# GEOTECHNICAL INVESTIGATION
# GUADALUPE RIVER FLOOD CONTROL IMPROVEMENTS
# UTHERN PACIFIC RAILROAD TO THE COUNTY MARINA
# IN ALVISO

San Jose, California

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# GUADALUPE RIVER FLOOD CONTROL IMPROVEMENTS
# SOUTHERN PACIFIC RAILROAD TO THE COUNTY MARINA
# IN ALVISO

San Jose, California

Prepared For

ANDERSON-NICHOLS & CO., INC.
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Palo Alto, CA 94303

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January 13, 1983

Project: 15482-V

Anderson-Nichols & Co., Inc.
2666 East Bayshore Road
Palo Alto, CA 94303

Attention: Mr. Arthur Woodworth

Gentlemen:

As requested, we have made a geotechnical investigation for the proposed Guadalupe River Flood Control Improvements near Alviso. This work was done in accordance with our proposal of September 27, 1982. The accompanying report presents the results of our investigation along with our opinions and recommendations for the design and construction of new levees and floodwalls for the project. Our opinions and recommendations were developed in consultation with representatives of Anderson-Nichols & Co., Inc. During the course of this work, we attended a public meeting in Alviso on November 23, 1982, to discuss the geotechnical design concepts.

The recommendations in this report are preliminary in nature and intended to assist in planning and preliminary design. When design details become established, our firm should be contacted to review plans and specifications and provide additional geotechnical recommendations as needed.

It has been a pleasure working with you on such an interesting and challenging engineering project. Please contact us if there are any questions regarding the contents of this report.

Sincerely,

William A. Bischoff

William A. Bischoff
Branch Manager

cam

# TABLE OF CONTENTS

Page

INTRODUCTION 1

PROJECT DESCRIPTION 1

SITE HISTORY 2

SITE AND GEOTECHNICAL FEATURES 6

- Surface Features 7
  - Station 10+00 to Station 20+00 7
  - Station 20+00 to Station 42+00 7
- Subsurface Conditions 9
  - Station 10+00 to Station 20+00 10
  - Station 20+00 to Station 42+00 11
- Groundwater and Seepage Conditions 14

PROJECT ALTERNATIVES 15

DISCUSSION AND RECOMMENDATIONS 16

- Levees 16
  - Static Stability 16
  - Seismic Stability 16
    - Design Earthquakes 17
    - Liquefaction Potential 18
    - Levee Stability and Deformations 19
  - Slope Inclinations 21
  - Set Back 21
  - Settlements 22
  - Subdrains 24
  - Construction 24
    - Stripping 24
    - Overexcavation and Keying 24
    - Materials and Placement 25
  - Scour and Erosion Protection 26
- Retaining Walls 26
  - Cantilever Walls - (Conventional) 26
    - Design Lateral Pressures 26
    - Foundation Support 27
    - Temporary Shoring 30
  - Soldier Pile/Slurry Wall or Lagging 31
- Reinforced Earth Walls 32
- Sheet Pile Walls 34
  - Design Lengths 34
  - Construction Considerations 34
- Seepage 35

TABLE OF CONTENTS
(continued)

Page

Groundwater Control 36

LIMITATIONS 37

APPENDIX A - FIELD INVESTIGATION AND LABORATORY TESTS

FIGURE 1 - SITE AND EXPLORATION PLAN

FIGURE 2 - IDEALIZED SOIL PROFILE

FIGURE 3 - BORING LOG LEGEND SHEET

FIGURES 4 THROUGH 17 - LOGS OF BORINGS

FIGURES 18 AND 19 - GRAIN SIZE DISTRIBUTION CURVES

FIGURE 20 - PLASTICITY INDEX CLASSIFICATION

FIGURE 21 - GUADALUPE RIVER IMPROVEMENT SUBDRAINS

APPENDIX B - PROJECT ALTERNATIVES

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## INTRODUCTION

In order to provide flood protection for the town of Alviso from high water levels in the Guadalupe River, plans are being prepared by Anderson-Nichols & Co., Inc. to improve the existing Guadalupe River levees. This work is being done under a contract with the Santa Clara Valley Water District.

As a member of the design team, Woodward-Clyde Consultants has made a geotechnical investigation for the project and has developed preliminary geotechnical design and construction recommendations to be used in assessing the technical feasibility of various flood control alternatives and in estimating their respective probable costs. The recommendations, as presented in this report, are based on engineering judgment as well as the results of the field investigation and laboratory testing.

## PROJECT DESCRIPTION

Levees along the Guadalupe River and flanking the western boundary of the town of Alviso are considered to be too low to provide adequate flood protection. It is understood that raising the existing levee system to Elevation 14 will provide protection from the 100-year floodwaters.

The project begins north of the existing Southern Pacific Railroad bridge near Gold Street and extends downstream along the Guadalupe River to the entrance of the County Marina.

The location of the project is shown on the attached Site and Exploration Plan, Figure 1. As presently conceived, flood protection will be increased by either raising existing levees or constructing new floodwalls. Conventional concrete retaining walls, reinforced earth or sheet piling are being considered for areas where floodwalls are needed.

Along the project, certain existing buildings are considered to have historical significance. These include the South Bay Yacht Club and remnants of an old cannery and icehouse, as shown on Figure 1. The proximity of these buildings to the east side of the existing levee presents horizontal constraints to the construction of new earth fill to provide a wider and higher new levee section. An existing concrete storage building located within the existing wildlife refuge area on the north end of the project is also within the path of potential new levee construction. Various design alternatives are being considered to preserve these structures.

Marshlands exist on the river side of the existing levees. In most areas, these marshlands are known to be the habitat of wildlife considered to be on the endangered species list. Therefore, construction of new levees in these areas is not being considered at this time.

## SITE HISTORY

Since Alviso's incorporation in the early 1850's, significant changes to the alignment of the Guadalupe River and Alviso Slough have been made. Prior to the 1850's, the original course of the Guadalupe River meandered to the west toward Guadalupe Slough and the San Francisco Bay. At that time, the Alviso Slough also skirted the northwest limits of Alviso. In order to establish a navigable waterway adjacent to the town, a channel was excavated along the western border of the town to connect the Alviso Slough and the Guadalupe River. Based on a discussion with Mr. Clyde Arbuckle, City of San Jose historian, it is understood that the man-made channel was completed in about 1858. Subsequently, the original course of the Guadalupe River extending west from the South Bay Yacht Club to the Guadalupe Slough was blocked by levee construction and the area is now used for salt evaporation ponds. Thus, the man-made channel along the west edge of Alviso became the Guadalupe River channel and the river now empties into the Alviso Slough.

As the development in the Santa Clara Valley increased during the 1900's, there was a resulting increase in the need for water. Withdrawal of groundwater from wells extending into deep aquifers resulted in consolidation of deep clay layers which caused a general subsidence of the ground surface across much of the valley including the Alviso area. Although records are not complete, it has been estimated by the United States Geological Survey (USGS) that areal subsidence in the Alviso area during the period from 1934 to 1967 was approximately 6 feet. The result of this subsidence was to lower the elevation of some areas adjoining the Guadalupe River below the high water levels in the river.

Based on a conversation with Mr. John Clayney, President of the South Bay Yacht Club, it is understood that the existing yacht club building (Figure 1) was constructed sometime during the period from 1907 to 1910. At the time of its construction, the building site and the other land adjoining the river channel were at a higher elevation than the water levels in the river. As the areal subsidence began to affect the town's susceptibility to flooding, it became necessary to construct levees along the river banks. It is understood that the first major effort was undertaken in the 1930's as part of a WPA project. Materials dredged from the channel area apparently were placed along the river banks. Land equipment was later used to raise the levees to even higher levels. By 1972, the ground subsidence had required that the levee near the yacht club be raised to a height of more than 8 feet above the adjoining street grade.

Beginning in the early 1900's, the Guadalupe River served as a transportation and shipping line for Alviso and the businesses which were located on its banks. Various enterprises which utilized the waterway for shipping included a cannery, a fuel storage tank farm, and a coal shipping/ receiving plant. Review of an aerial photograph from 1957 indicates that the cannery buildings actually extended to the rivers edge as approximately shown on Figure 1. The locations of the remaining portions of the cannery buildings, the previous tank farm dock, piping and tank foundations are also shown on Figure 1. Based upon a discussion with Mr. Garland Oliver, long time resident of Alviso, it is understood that demolition of portions of the cannery buildings along the river's edge was necessary in the early 1960's to allow for the placement of levee fills along that section of the east bank. That levee fill actually bears against some of the remaining walls of the existing cannery structures.

Between about Stations 10+00 and 20+00 of the current project, the river was straightened from the meandering course which existed prior to 1965 to its current alignment. The realignment was accomplished by dredging and the materials generated from the process were placed as fill in the previous channel to create a broad levee between the new river alignment and the railroad. Plans for this realignment dated March 12, 1965 indicate that the planned top of levee was Elevation 13. In conjunction with this river straightening, the existing railroad track was realigned and raised. A new bridge crossing the river was constructed just beyond the south end of the current project limits. Later, new mooring facilities for the boats of the South Bay Yacht Club were installed to the west of the levee as shown on Figure 1. Based on a discussion with Mr. Clayney, it is understood that a 500 gallon gasoline storage tank was buried within the levee between about Stations 22+00 and 23+00 of the project alignment. Also during the latter part of the 1960's, existing salt pond levees were realigned and a County Marina constructed on an extension into the reclaimed area at the north end of the current project limits.

The alignment of the Guadalupe River and topographic features of the surrounding area shown on Figure 1 are those which existed during the early 1970's. Based upon our review of more recent topographic information, it appears that only minor alterations have been made to the levee area in recent years. Most of the work during this time has been located on the section of levee across from the South Bay Yacht Club building. In January 1973, a portion of a timber retaining wall along the base of the levee adjacent to Hope Street failed during a high tide and an emergency fill was placed by the Santa Clara Valley Water District to buttress the levee. However, the fill blocked the street to vehicle traffic and was subsequently removed. In January 1978, isolated problems associated with the levee developed during another high tide. These problems, as reported in a memorandum by Mr. J. D. Cooper of the Santa Clara Valley Water

District dated February 14, 1978, included a small "sinkhole" of about 2 feet in diameter located in the levee fill at the west end of Catherine Street and a small slide in front of the yacht club building which displaced about one cubic yard of saturated material. Clear water was reported to have leaked at a rate of up to 5 gpm at the highest noted tide level. The flow through the levee diminished as the tide level fell. Subsequent to these problems, a gabion wall was constructed against a portion of the levee in front of the yacht club building which included the area which experienced the sloughing.

Recently, additional soil materials have been placed to top off some of the lower areas along the levee. Mr. David M. McNeeley of the Santa Clara Valley Water District has indicated that this was done to provide a minimum levee crest of about Elevation 9.

During a high tide of November 30, 1982, sand bags were placed along the top of the levee near the gasoline pump dock area of the yacht club facilities, and soil and rubble materials were dumped at several locations between about Stations 35+00 to 38+50 to provide protection against overtopping during the high tide and waves.

## SITE AND GEOTECHNICAL FEATURES

For the purpose of the following discussion, the alignment has been subdivided into two sections corresponding to those reaches along which distinctly different surface and subsurface conditions have been encountered. The first section extends from Station 10+00 to Station 20+00; the second section covers the area from Station 20+00 to the end of the alignment at Station 42+00.

## Surface Features

Station 10+00 to Station 20+00 - This section of the alignment is characterized by a broad, uneven levee which is covered with low grasses and vegetation on either side of the levee and over parts of the crest. Some concrete rubble, wood, and other debris are present on the ground surface near the south end of the levee.

Topographic information developed in 1982 indicates that the levee crest ranged from about Elevation 7-1/2 to Elevation 10-1/2 in this area prior to its recent topping off to Elevation 9. Slope inclinations on the inward side of the levee range from about 3 to 1 (horizontal to vertical) to 6 to 1 or flatter. On the river side of the levee, the slopes are about 4 to 1 until the tidal zone is reached. In the tidal zone, soft sediment has been deposited at very shallow slopes which extend to the edge of the river channel. South of Station 20+00, this zone is up to 200 feet wide due to deposition behind the mooring facilities of the South Bay Yacht Club. Bullrush and other tidal marsh plants cover much of this shallow sediment. There are also numerous animal burrows located in the upper levee areas in the north part of this section.

Station 20+00 to Station 42+00 - North of about Station 20+00, the levee begins a transition to a narrower section with steeper side slopes which characterize much of the remainder of the alignment. Extending from the levee crest down to the tidal zone, the side slopes are generally inclined at about 2 to 1 (horizontal to vertical) to 3 to 1; although along some sections, the top 3 to 5 feet stand at inclinations of 1/2 to 1 or steeper. The tidal sediments north of Station 20+00 are much narrower in width, but remain shallow in slope and are present over most of the alignment. There are numerous minor boat docking facilities between Stations 21+00 and Station 35+00. Other notable features on the outer levee side include a timber pile and concrete bulkhead at the end of Catherine Street at about Station 25+50 and a boat launching ramp near Station 33+00 which is no longer in use and has filled with sediment.

Topographic information indicates that the levee crest over the remainder of the alignment generally varied between about Elevations 8 and 12 prior to the placement of additional fill to top off the levee in November, 1982. Based on visual observations, it appears that the fill was placed between approximately Stations 20+50 to 25+00 and Stations 32+00 to 41+00. As previously discussed in "Site History," some soil and concrete rubble also was placed more recently between about Stations 35+00 and 38+50 to prevent overtopping of the levee during an extreme high tide.

The inboard levee slopes are somewhat more variable. In the vicinity of the yacht club building, the slopes range from about 2 to 1 to nearly 1 to 1. The gabion wall is present from about Station 21+75 to Station 22+25. North of the gabion wall, the slopes gradually flatten to about 2-1/2 to 1 and remain at that inclination until about Station 28+00, with the exception of the flatter access to the Catherine Street bulkhead. These slopes are covered with low grasses and plants with some loose debris in places. At about Station 28+00, the inboard levee slopes become steeper and encroach several feet upon the corner of the existing portion of the icehouse structure. North of Elizabeth Street, the inboard levee slopes actually bear against the remaining walls of the existing cannery buildings in several areas, most notably from Station 30+75 to Station 32+50. In this area, the existing levee crest is fairly level and extends to the limits of the existing building walls. This section of the levee is littered with debris including barrels, rugs, concrete, wood, and discarded equipment.

At about Station 33+00, the alignment turns west and continues around the perimeter of a parcel owned by the Fish and Wildlife Department. Prior to the recent fill placement, this area was characterized by fairly gentle inboard levee slopes. At present, the inboard and river side levee slopes are relatively steep and tension cracking is evident along the top of the slope. Even prior to the recent soil and rubble fill placement, concrete rubble was observed to be exposed in the levee side slopes in this area.

From the Fish and Wildlife area, the alignment runs along a fill section leading to the County Marina parking lot. The interior side of this levee section slopes down to a pond which was once part of a slough before the construction of the marina closed it. The water level in the pond is near Elevation 0 and the surface around the perimeter of the pond is covered with scattered concrete rubble and other debris.

## Subsurface Conditions

Fourteen exploratory borings which ranged in depth from 11-1/2 feet to 45 feet were drilled for this investigation at the approximate locations shown on Figure 1. In order to generalize the subsurface conditions encountered along the alignment, an "Idealized Soil Profile" has been prepared and is presented as Figure 2. More comprehensive descriptions of the subsurface conditions encountered at each boring location are shown on the Logs of Borings presented as

Figures 4 through 17. Discussion of the subsurface conditions has again been divided into two sections.

Station 10+00 to Station 20+00 - As discussed in "Site History," the original course of the Guadalupe River was straightened in 1965 and the materials which were dredged to provide the present course were placed as fill to create the existing levee along this section of the alignment. In Borings 6 and 7, the levee fill extending from the crest to about Elevation 2 was found to consist primarily of high plasticity clays which contain some sandy zones and occasional wood fragments. These shallow levee fills are of moderate strength and could conceivably be either dredged materials which have gained strength through drying and dessication or possibly materials which were placed using land equipment.

Below the surficial fill and extending to as deep as Elevation -32 in Boring 6, the soils were found to consist primarily of a soft, dark gray and black silty clay which is known locally as Bay Mud. The Bay Mud is of low density, high moisture content, and is considered to be of relatively high compressibility. Seams and lenses of fine sand as well as traces of organic material were found throughout the layer and the bottom few feet in each of the borings was found to grade to a coarser sand in a clay matrix. In addition, fragments of glass, wood, coal and ceramic tile were recovered from various boring depths. The presence of these materials supports the conclusion that the soils within this depth interval are mostly dredged fill or are a combination of dredged fill overlying soft channel deposits. In any event, the engineering properties of the layer do not vary greatly with depth and it is considered likely that the layer is near the end of its primary consolidation since being placed and, thus, is probably approaching a normally consolidated condition.

The native soils found below Elevation -25 in Boring 7 consist of a layer of stiff sandy clay which was underlain by medium dense to dense sandy silt extending to the bottom of the boring at about Elevation -32. Native soils in Boring 6 were found below about Elevation -32 and consist of dense sand to the limit of the depth explored at Elevation -33-1/2. The native soils found in the borings in this section were similar to the deeper soil conditions logged at approximately similar elevations in Boring R-1 drilled as part of the "Soil Investigation for the Proposed Guadalupe River Project - Reaches 1 and 2" prepared by our predessor firm, Woodward-Clyde-Sherard & Associates in 1960. The general subsurface conditions logged on that boring have been included on the "Idealized Soil Profile" presented as Figure 2. The approximate location of Boring R-1 also is shown on Figure 2.

Station 20+00 to Station 42+00 - The soils along this section of the proposed alignment can be generalized as consisting of heterogeneous mixtures of fill overlying a varying thickness of Bay Mud which is, in turn, underlain by layers of alluvial clays and sands. The fill sections encountered in the borings generally extend to within a few feet above or below Elevation 0. From Station 20+00 to approximately Station 28+00, the fill section was found to consist primarily of silty clay and clayey gravel with zones of silty sand. With the exception of the upper six or seven feet of moderately compacted fill encountered in Boring 5 (located on the levee in front of the yacht club), most of the fill is considered to be in a poorly compacted condition. Most of the clay fill is highly plastic and has areas of high concentrations of shells, rock fragments, and sand. In Borings 4 and 5, approximately the lower three feet consist of a dark gray to black sandy clay with shell fragments. It is considered likely that this material represents a dredged Bay Mud fill placed during the initial levee construction.

North of Station 28+00 and to about Station 32+00, the fill also extends to near Elevation 0. In this area, the existing levee fill consists of a heterogeneous mixture of soils containing rock fragments, asphalt, broken concrete and brick. It is likely that some of these rubble materials might have been generated from the demolition of the portions of the cannery and icehouse building which previously occupied the area. Borings 9 and 10, which were located within the limits of the old cannery building footprint shown on Figure 1, each encountered hard drilling through a section of concrete about 4 inches thick at about Elevation -2. It is considered probable that the concrete is part of the floor slab of the demolished building section. Although Borings 3 and 11, which also were located within the old building limits, encountered only pieces of broken concrete, it is likely that a large portion of the existing levee fill in this area could be underlain by old concrete slabs and building foundations.

Beyond about Station 32+00, the fills encountered in Borings 1 and 2 were found to consist of poorly compacted mixtures of clays, sands and gravels. As previously noted, pieces of broken concrete were observed in the side slopes of the levee but none were encountered in the borings. It appears that the lower fill layers around the Fish and Wildlife parcel also may have consisted of dredged Bay Mud similar to conditions found in the south portion of this section. In

Boring 8, which was drilled through the levee section leading to the County Marina parking lot, the fill conditions were found to be considerably different than those encountered in Borings 1 and 2. The levee fill consists of moderately to well compacted layers of silty clay extending to about Elevation -10.

As shown on the "Idealized Soil Profile," Figure 2, the fill along most of this section of the alignment is underlain by Bay Mud. The thickness of the Bay Mud layer varies; however, the base of the layer generally extends to about Elevation -10. The Bay Mud is typically soft and is considered to be highly compressible; although, the lower few feet of the layer encountered in some of the borings exhibit somewhat higher strengths. Thin sand seams, sand lenses, along with some shells and wood pieces also were found in the layer. The Bay Mud layer in Borings 3, 9, 10 and 11, which were located further from the water's edge, was only a few feet thick or not encountered at all, which suggests that the layer decreases in thickness as the distance from the river increases. In Boring 8, it appears likely that most of the Bay Mud is underlain by alluvial deposits consisting of silty to sandy clays and fine to medium grained sands. The clays are stiff to very stiff in consistency and the sands are typically medium dense. These sand and clay layers were found to extend to the limits of the borings drilled along this section of the alignment.

It should be emphasized that the paragraphs presented above reference average strata depths and generalized material descriptions. Significant variations from these generalized descriptions do occur from one boring to the next. In addition, the lines designating the interface between the soil strata on the idealized profile and on the Logs of Borings represent approximate boundaries. The depths to the indicated interfaces should be considered approximate and the transitions between the materials to be gradual.

### Groundwater and Seepage Conditions

Groundwater was measured in the borings drilled for this investigation at the time of drilling or within a period of about one week after the end of drilling. The water levels measured in the borings made for this investigation are indicated on the Logs of Borings, Figures 4 through 17 and the highest measured water levels are noted on the "Idealized Soil Profile," Figure 2.

In general, the groundwater level was found to vary from Elevation +5-1/2 to Elevation -5 at various points along the alignment with the water generally measured above Elevation 0 in those borings made along the levee crest and below Elevation 0 (or slightly below the existing grade) in those borings made beyond the inboard toe of the levee. Additionally, water level measurements made in borings which were found to be open on December 2, 1982 (following the high water level of November 30, 1982) indicated a groundwater rise of 1 to 2 feet in most of the borings.

During periods of high water in the river and, to a lesser extent, at most times, wet areas have been observed along the inboard toe of the levee which are suspected to be due to seepage through the levee section. Areas where seepage was observed on December 2, 1982 are shown on Figure 1 and included a small flow under the south corner of the gabion wall near the yacht club building, ponded water along the southwest corner of Hope and Catherine

Streets at the toe of the levee, ponded water at the toe of the levee north of the Catherine Street bulkhead, and along the retaining wall supporting the toe of the levee above the existing cannery building slab at about Station 29+50. Discussions with residents of the area indicate that seepage has been observed along much of the previous buildings limits at various times and it is surmised that the seepage could be occurring along the interface between the base of the levee fill and the concrete slab which is suspected to be present in this area. Abandoned piping networks which extend through the levee section, such as those associated with the old fuel tank farm, animal burrows, or simply pervious zones in the levee fill could also be serving as conduits for levee seepage.

## PROJECT ALTERNATIVES

Anderson-Nichols & Co., Inc. have developed three alternative concepts to achieve a goal of flood protection to Elevation 14 along the portion of the Guadalupe River from the Southern Pacific Railroad bridge to the County Marina. Plans showing the alternatives are attached to this report as Appendix B.

Alternative A consists of a system of levees and floodwalls extending along the entire reach for the subject project. The floodwalls are intended to be used principally to preserve historic structures. Alternative B is also a system of levees and floodwalls but bypasses the fish and wildlife preserve at the north end of the project. Alternative C is almost exclusively a system of levees, some of which would encroach on several of the historic buildings. With this alternative, only one retaining wall would be constructed to preserve the South Bay Yacht Club.

For each alternative, various reaches along the channel have similar surface conditions or horizontal constraints and, therefore, similar solutions to the flood improvement problems are being proposed. Reaches with similar conditions are identified as Section 1, Section 2, and Section 3. For each section, there are at least two alternate solutions. The alternates shown in Appendix B along with other possible retaining wall systems are discussed in the following section of this report.

## DISCUSSION AND RECOMMENDATIONS

### Levees

Static Stability - In order to form opinions regarding the potential effects of constructing new levee fills along the project alignment, slope stability studies have been performed. Computer assisted slope stability calculations were made using both the ordinary method of slices as well as Bishop's modified method of slices. These calculations were performed assuming soil strength parameters, fluctuating tide levels and existing levee and river bank profiles which are assumed to be generally representative of the conditions which exist between Stations 10+00 to 20+00 and from Stations 20+00 to 42+00. Based upon the results of the calculations, as well as engineering judgment, it is expected that the foundation materials will be capable of supporting the static loads of the new levee fills.

Seismic Stability - A study has been made of the seismic stability and estimated deformation of the levees for selected design earthquakes. Estimates of levee deformation during earthquake shaking have been based upon techniques developed by Seed and Makdisi (1) used in conjunction with the computer assisted stability analysis discussed above. The study also has included an assessment of liquefaction potential at the site and its influence on levee stability. The assessment of the liquefaction potential along the project alignment was based upon recent analytical techniques developed by Seed and Idriss (2) as well as engineering judgment.

Design Earthquakes - The following four design earthquakes were considered for the studies:

| Earthquake Source | Earthquake Magnitude, M |
|---|---|
| San Andreas fault | 8-1/4 |
| (13 km from site) | 7-1/4 |
| Hayward fault | 7-1/2 |
| (5 km from site) | 6-1/2 |

The larger magnitudes listed above for each fault (M = 8-1/4 on San Andreas and M = 7-1/2 on Hayward) represent estimated maximum credible earthquakes (i.e. the largest earthquakes that appear capable of occurring on the faults with our current knowledge of the tectonic framework). The smaller magnitudes M = 7-1/4 on San Andreas and M = 6-1/2 on Hayward) represent estimates of 100-year earthquakes (i.e. earthquakes that appear

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[1] Makdisi, F.I., and Seed, H.B., "Simplified Procedure For Estimating Dam and Embankment Earthquake-Induced Deformations" Journal of the Geotechnical Engineering Division, ASCE Vol. 104, No. GT7, July 1978.

[2] Seed, H.B., and Idriss, I.M., "Evaluation of the Liquefaction Potential of Sand Deposits Based Upon Observations of Performance in Previous Earthquakes" paper presented at the Symposium on In-Situ Testing to Evaluate Liquefaction Potential, ASCE National Cenvention, St. Louis, Missouri, October, 1981, Preprint 81-544.

to have recurrence intervals of about 100 years, on the average, on the faults). Each of these design earthquakes was assumed to occur on the portion of the fault closest to the site. It is judged that earthquakes on the San Andreas and Hayward faults are more critical for levee stability at the site than earthquakes on other known active faults in the site region.

Liquefaction Potential - Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. The new levee fills will be above the groundwater table and thus would not be expected to be susceptible to liquefaction. The existing levee fills are, in general, quite clayey and also would not be expected to liquefy. Some portions of the existing levees do contain loose sands below the groundwater table that could liquefy during any of the design earthquakes considered; however, these portions do not appear to be widespread.

The dredged Bay Mud fill encountered between Stations 10+00 and 20+00 contains numerous sand lenses interbedded with soft clays. The sand lenses are probably loose and may be expected to liquefy during the design earthquakes. The natural Bay Mud which underlies the fill from about Station 20+00 to 42+00 also contains sand lenses which could liquefy. However the lenses in the natural Bay Mud as noted on the Logs of Borings appear to be very localized in comparison to those in the dredged Bay Mud fill.

Sand lenses and layers interbedded with stiff clays were encountered in the alluvial deposits beneath the Bay Mud and dredged Bay Mud fill. Some of these sands appear to be loose enough to liquefy during the design earthquakes.

Levee Stability and Deformations - The study of seismic levee stability and deformation has considered the potential for liquefaction in sands, as described above, as well as the potential for shear failure in the soft clays of the natural and dredged Bay Mud deposits, and the underlying stiffer clays. The estimated behavior described below is for earthquakes on the San Andreas fault, which were found to be more critical for levee stability than earthquakes on the Hayward fault.

During maximum credible earthquakes, portions of the levee underlain by natural Bay Mud deposits with thicknesses of about 4 feet or greater (such as at Boring 1 where the Mud is about 12 feet thick) could experience from about 5 feet of horizontal displacement to complete levee failure. During the 100-year earthquake, it is estimated that horizontal levee displacements might range from less than 1 foot to as much as 6 feet. For portions of the levee underlain by relatively thin (less than about 4 feet) layers of Bay Mud, or where the Mud is absent, horizontal displacements are estimated to range from less than 1 foot to 3 feet during the maximum credible or 100-year earthquake.

Where the new levee fill would be underlain by thick deposits of dredged Bay Mud fill (such as at Boring 6 where 34 feet of Bay Mud fill were encountered), the behavior may range from small displacements to complete failure during both maximum credible and 100-year earthquakes due to liquefaction in the sand lenses within the Bay Mud fill combined with shear failure in the soft clays of the fill.

It is expected that some slumping of the levee also would occur in conjunction with the horizontal displacement. It is estimated that the magnitude of slumping might be of the order of about one-half of horizontal displacements discussed above. Areas where the existing levee contains substantial zones of saturated sands could experience complete levee failure during the design earthquakes due to liquefaction of the sands. However, as noted previously, the sands do not appear to be widespread in the fill, and thus any such failures are likely to be localized.

Although the alluvial sands below the Bay Mud are susceptible to liquefaction as discussed previously, liquefaction of these deeper sands is not expected to affect levee stability. Rather, stability is expected to be governed by the behavior of the natural Bay Mud, dredged Bay Mud fill, and sand zones within existing levee fills.

Geotechnical studies leading to recommendations for improving levee seismic stability are beyond the scope of this report. However, considering the depth and lateral extent of Bay Mud as well as the undefined location of sand lenses within the existing fill, any seismic levee stability improvement is likely to be extensive. Consideration should be given to the likely benefit/cost ratio of such levee improvements before additional design studies are made. The low probability of a levee damaging earthquake occurring at the same time as high flows in the Guadalupe River should also be considered.

Slope Inclinations - The maximum slope inclinations for the sides of the levees are dependent upon the thickness of fill to be added as well as the existing soil conditions. Based upon the subsurface conditions encountered in the exploratory borings drilled for this study and our understanding of the levee construction planned at this time, it is recommended that the maximum side slopes for both the inboard and outboard faces of the levee be designed for inclinations not steeper than 2 to 1 (horizontal to vertical). This recommended levee slope is predicated on the assumption that the areas to receive new levee fill will be properly prepared and that the fill will be constructed to the requirements of engineered fill. Recommendations regarding the preparation for new fill and its placement and compaction are discussed in a following section entitled "Construction."

Set Back - New levee fills from about Station 21+00 to the County Marina should be set back from the outboard top edge of the existing levee. It is recommended that a set back distance of 10 feet be maintained where new levee fills are planned around the Fish and Wildlife refuge area (approximate Stations 33+00 to 40+00). At other locations from Stations 21+00 to 32+00, a 5 foot set back distance should be used. The intent of this recommended set back distance is to help minimize the potential for undermining of the toe of the new levee fill due to the weak or oversteepened nature of the outboard face of the existing levee.

Settlements - Fill placed to raise the existing levees will cause compression of the existing Bay Mud deposits and to a lesser degree, compression of the existing poorly compacted fills. The amount of settlement to be anticipated is dependent upon the thickness of new levee fill as well as the thickness of the existing compressible deposits.

The settlement estimates, as presented in the table below, are based upon the subsurface conditions encountered in the exploratory borings, information gathered about site history, an understanding of the new proposed levee fill thicknesses and engineering judgment.

## ESTIMATED LEVEE SETTLEMENTS

| | Station Location (1) | | | | |
|---|---|---|---|---|---|
| | 12+00 to 19+00 | 19+00 to 23+00 | 23+00 to 28+00 | 28+00 to 33+00 | 33+00 to 40+00 |
| Proposed new (2) fill thickness (feet) | 4-6 | 14 | 10 | 3-6 | 5-9 |
| Existing fill (2) thickness (feet) | 5-6 | 12-15 | 4-5 | 10-12 | 5-12 |
| Bay Mud thickness (feet) | 25-34 | 0-5 | 8-10 | 0-4 | 10-12 |
| Estimated Ultimate settlement (in.) | 15-20 | 12 | 10-12 | 1-2 | 14-17 |
| Estimated Time for 50% settlement (yrs) | 4-7 | ½ | ½-1 | ½ | ½-1 |
| Estimated Time for 90% settlement (yrs) | 15-25 | 1-2 | 3-5 | 1-2 | 3-5 |

(1) Refer to Site and Exploration Plan (Figure 1).

(2) Section of maximum new fill thickness may be offset from center line. Thicknesses may not correlate to those shown on Figure 2.

It should be recognized that the actual settlements will be dependent also on certain unknown conditions including variations in Bay Mud thicknesses and the compressibility of the existing fill.

Settlement of the levees will occur gradually. Estimates of time for settlement to occur are presented above. The rate of settlement is dependent upon the thickness and permeability of the compressible layer as well as the location of sand lenses within the deposit which are difficult to identify. The ranges of time estimates for each category are attempts to bracket uncertainties of the site conditions.

To compensate for the ultimate settlement of the levee, additional fill could be added at the time of new levee construction. The thickness of fill added to compensate for the settlement should equal about 120 percent of the estimated settlement for that particular reach. For example: if it is estimated that 20 inches of settlement will occur for a particular reach of levee, an additional amount of 24 inches of fill should be added to provide an initial top of levee of Elevation 16. It is estimated that this will result in an ultimate top of levee of about Elevation 14.

Alternatively, the levees could be constructed to Elevation 14 at this time. The rate of settlement could be monitored and additional fill added to top off the dikes as settlement takes place. The initial levee top width should be sufficient to provide room for future filling.

Subdrains - It is recommended that subsurface drains be installed at the interface between existing levees and new levee fills. The drains should be used to intercept existing seepage. As shown on Figure 21, the drains should consist of a chimney of Class 2 Permeable Material containing a 4-inch diameter, perforated pipe. Each pipe should be sloped to drain toward a sump or storm drain system. The toe of the existing levee should be excavated to allow for the drain installation. The extent of excavation and subdrain installation should be determined by the Geotechnical Engineer during construction.

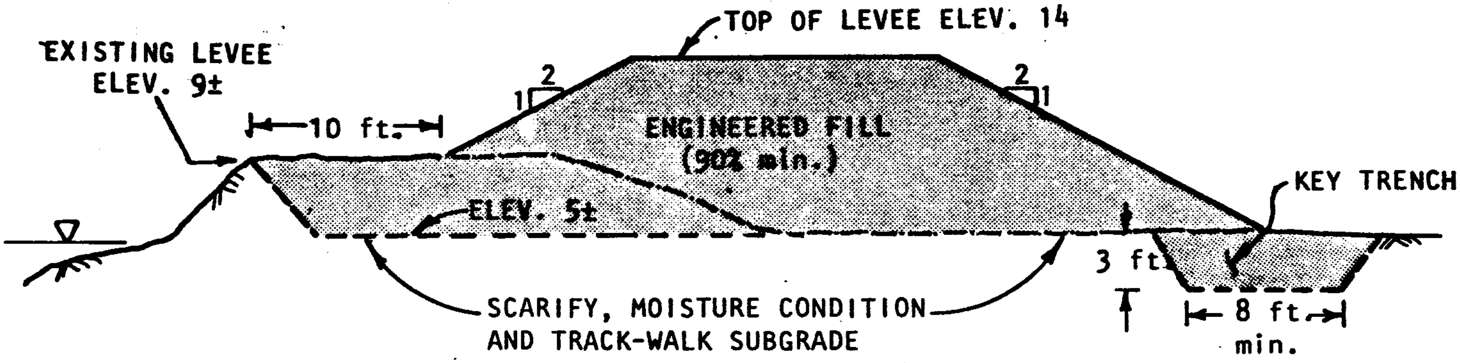
Construction -

Stripping - Before any fill is placed to construct levees, the existing site surface should be properly prepared. Any existing organic material or loose rubble deposits should be removed. In most of the areas, it is expected that stripping depths of the order of 6 to 12 inches might be necessary. The final depth of stripping and removal of loose fill should be approved by the Geotechnical Engineer during construction.

Overexcavation and Keying - After stripping has been completed, key trenches should be excavated at the proposed inboard and outboard toes of the new levee fill. The trenches should be at least equipment width (8 feet±) and extend to a minimum depth of 3 feet below the existing grade. The exposed surface should then be moistured conditioned and compacted by "track-walking."

All other areas to receive levee fill should be moisture conditioned and compacted as recommended above.

In the vicinity of the Fish and Wildlife Preserve, (Stations 33+00 to 40+00), the surface of the existing levee contains pipes, broken concrete, and recently placed loose fill. The fill is sloughing on both sides of the levee. Because of the condition exposed on the levee surface and the probability that similar materials are buried within the levee, it is recommended that the levees be excavated to Elevation 5 as shown on the sketch below. The base of the excavation should be scarified, moisture conditioned and track-walked prior to placing new levee fill.



Materials and Placement - Material used for levee construction should be a soil or soil-rock mixture. The material should be comprised primarily of clayey soils of low to medium plasticity as classified by the Unified Soil Classification System. The clay should have a plasticity index within the range of 10 to 25. Sand and gravel size particles should comprise no more than 50 percent of the soil-rock mixture. All fill placement and compaction should be done in accordance with the applicable sections of the Standard Specifications of Caltrans, latest edition. Levee fill should be compacted to a minimun degree of compaction of 90 percent.

Scour and Erosion Protection - Scour and erosion protection should be provided where high flow velocities or wave action could cause erosion of the slope surface. The slope surface should be examined periodically and repaired as necessary to maintain the design embankment section.

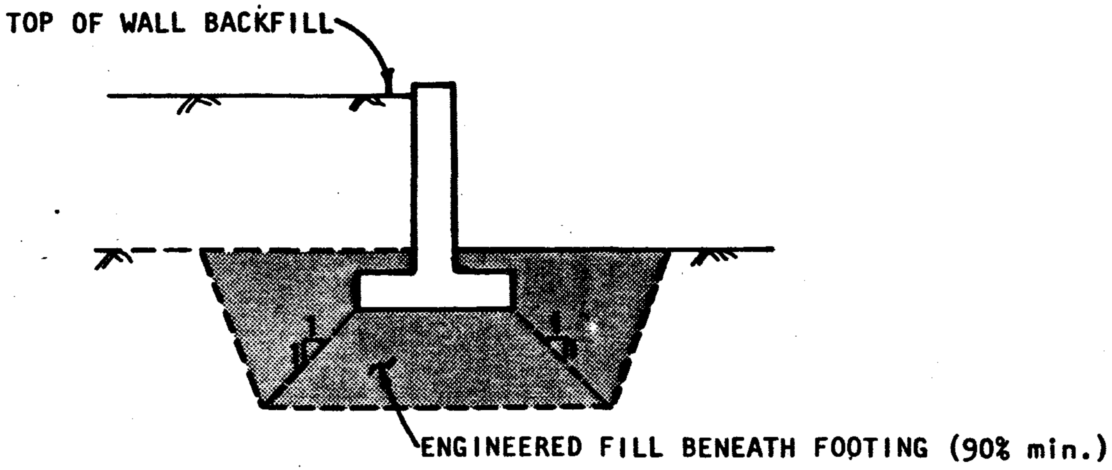
### Retaining Walls

#### Cantilever Walls - (Conventional)-

Design Lateral Pressures - Cantilever walls should be designed to resist long-term equivalent fluid pressures of 90 pcf. This assumes a combination of soil and hydrostatic pressures are bearing against the wall. If traffic or other surcharge loads are expected near the back of the wall, the walls should be designed to resist these loading conditions. Auto and light truck traffic loads can be simulated by assuming a 2 foot surcharge as being placed on the back of the wall. For other surcharge loads, a uniform horizontal pressure equal to 1/2 of the anticipated surcharge loads should be applied to the back of the walls for design purposes. If exceptionally large surcharge loads are anticipated, the Geotechnical Engineer should be consulted for additional recommendations.

Low retaining walls (3 to 4 feet±) are being considered near the outside edge of fills proposed near the intersection of Hope and Catherine Street. The walls will support sloping backfill. Because of the low height of the walls as well as their distance from the river, it is considered that the walls can be drained with subdrains placed behind the walls. Consequently these walls can be designed to resist active soil pressures without hydrostatic pressures. For these conditions and assuming sloping backfill inclinations of 2 to 1 (horizontal to vertical), it is recommended that the walls be designed to resist equivalent fluid pressures of 55 pcf. The walls should be backfilled with Class 2 Permeable material as defined in Section 68 of Caltrans Standard Specifications. Also the face of the walls should contain weep holes at 5 feet center-to-center horizontal spacing. Alternatively, a perforated 4-inch diameter pipe could be installed near the base of the filter material and sloped to drain to the storm drain system. The top 1 foot of wall backfill should be a relatively impervious clay compacted to a minimum degree of compaction of 90 percent.

Foundation Support - Conventional cantilever walls can be supported on footing foundations bearing on engineered fill used to replace existing weak fills or soft, compressible Bay Mud deposits. Spread footings should be supported on a minimum thickness of engineered fill equal to the width of the retaining wall foundation. In areas where the combined thickness of existing fill and Bay Mud immediately below the proposed bottom of footing is less than the recommended thickness of new fill, the new fill thickness can be reduced to equal that of the combined existing fill and Bay Mud thickness. The fill should extend laterally beyond the toe of the wall foundation as shown schematically on the sketch below.



Footings bearing on engineered fill should be designed so that maximum bearing pressures do not exceed 3,000 psf due to dead loads, 4,000 psf due to dead plus live loads and 5,000 psf due to all loads including wind or seismic. Resistance to sliding can be assumed to be developed by a combination of friction along the base of the wall footing and passive pressures acting against the face of the footing and embedded portion of the wall. For design purposes, a coefficient of friction of 0.40 can be assumed along with a passive equivalent fluid pressure of 200 pcf.

In the areas of the South Bay Yacht Club and the old icehouse, tall retaining walls (10-15 feet) will be needed. Because of the large active and hydrostatic pressures to be retained, it will be extremely difficult, if not impossible, to resist wall sliding by either passive pressures, friction beneath the footing or keys. In these cases, additional resistance to sliding could be provided by drilled pier foundations. The piers should be designed to resist horizontal forces through pier bending-soil interaction. Estimates of lateral load capacity and maximum moments developed for 2 and 3 foot diameter piers are tabulated below for a variety of depths.

PIER DIAMETER = 2 FEET

| Pier Depth (ft) | Lateral Load Capacity (kips) | Maximum Moment Developed (ft-kips) |
|---|---|---|
| 5 | 10 | 40 |
| 10 | 35 | 230 |
| 15 | 60 | 550 |

PIER DIAMETER = 3 FEET

| Pier Depth (ft) | Lateral Load Capacity (kips) | Maximum Moment Developed (ft-kips) |
|---|---|---|
| 5 | 20 | 95 |
| 10 | 55 | 400 |
| 15 | 100 | 950 |

In estimating the lateral load capacities and moments tabulated above, it has been assumed the piers would be relatively fixed at the bottom of pier cap and restricted from rotation.

For vertical loads, the piers can be assumed to resist loads through peripheral skin friction acting between the pier shaft and the surrounding soil. A skin friction of 450 psf should be assumed for design purposes.

Temporary Shoring - Since horizontal constraints might require that retaining walls be positioned at locations which are well within the present levee section, the need for temporary shoring prior to excavation and retaining wall construction should be anticipated. For the construction of conventional cantilever retaining walls, temporary shoring could consist of steel sheet piling driven through the levee and Bay Mud layer to develop its support capacity in the underlying native stiff clays. The required depth of sheet pile embedment is a function of the embankment section and loading to be retained, the pile section selected and the embedment conditions. For preliminary purposes, it is recommended that an equivalent fluid pressure of 90 pcf be used for temporary sheet pile design. As for retaining wall design, this equivalent fluid pressure assumes full hydrostatic pressure and horizontal backfill conditions. For a bottom of pier cap excavation extending to about Elevation -5 at both the yacht club and icehouse wall locations, and for a levee top at Elevation 6 (expected minimum construction top of levee condition), it is estimated that the required sheet pile embedment might be of the order of 10 to 11 feet below the bottom of the Bay Mud layer or to approximate Elevation -18. This estimated embedment depth is based upon the use of either a PZ-27 or PZ-32 sheet pile section.

It is suggested that this estimated sheet pile embedment length be used for preliminary cost estimating purposes in those areas where temporary shoring for the construction of retaining walls is anticipated. More detailed design recommendations regarding the design of temporary shoring should be developed when wall locations, foundation depths and embankment configurations have been finalized.

Soldier Pile/Slurry Wall or Lagging - As an alternative to a conventional cantilever wall, soldier piles could be used in conjunction with either a slurry wall or timber lagging. Either the slurry wall or the timber lagging could be finished with a facing for aesthetic purposes. The wall should be designed to resist lateral soil and water pressures as recommended for conventional walls. The soldier pile foundations should consist of drilled cast-in-place concrete piers, reinforced with steel H-beams. Preliminary estimates of pier lateral load capacities and corresponding maximum moments developed for various pier diameters and depths are presented below. The parameters presented assume the piers are free to rotate at the ground surface.

PIER LATERAL LOAD CAPACITY (kips)

| | Pier Diameter | | |
|---|---|---|---|
| | 3 ft. | | 4 ft. |
| Pier Embedment Depth (ft.) | 25 | 30 | 25 |
| Lateral Load Capacity (kips) | 75 | 100 | 100 |
| Maximum Moment (ft-kips) | 950 | 1,300 | 1,300 |

After the piers are installed, the walls between the piers can be constructed. As discussed above, the wall could be constructed by either slurry techniques or with timber lagging. Both concepts offer the advantage of providing temporary lateral support to the existing levee during construction as well as long-term support.

It should be realized that the design criteria presented above for drilled piers used to support either conventional cantilever walls or soldier pile walls are approximate and based on general correlations of soil strength and lateral load capacity. The values are intended for use in preliminary conceptual design and cost estimating. Final design values should be established considering pier deflections which can be estimated through soil-pier interaction analysis. Such studies should be made during the final design stage after wall design details are finalized.

### Reinforced Earth Walls

Reinforced earth walls should be designed to resist the same combined lateral earth and water pressures recommended above for conventional cantilever walls. Reinforced earth is a concept of restraining vertically stacked concrete panels with steel straps extending horizontally into the wall backfill. As discussed with Anderson-Nichols & Co., Inc., this wall concept appears particularly applicable for various areas of the project as a part of a new levee fill and wall combination. One such area is in the vicinity of Catherine Street to Mill Street from approximately Station 29+00 to Station 33+00. In this area, approximately 3 feet to 6 feet of fill would be needed in conjunction with a reinforced earth wall. It is estimated that the addition of 3 feet to 6 feet of fill in this area might develop settlements of about 1 to 2 inches, as previously presented on page 22. Higher reinforced earth walls also are being considered in the area of the South Bay Yacht Club and the Fish and Wildlife Preserve. Settlement estimates for reinforced earth walls in these areas are expected to be the same as presented in the table on page 22 for embankment fill settlements.

The reinforced earth wall has the advantage of being able to accommodate differential settlements of as much as 0.1 feet in 10 feet of wall length which might be expected at this site.

The Reinforced Earth Company designs and supplies materials for reinforced earth structures. They are presently the sole licensee of the patents registered in the United States. Their services include design, preparation of plans and specifications for the structure, and the supply of the manufactured wall components.

The plans and specifications are normally prepared by the Reinforced Earth Company for the contracting agency. However, some agencies prefer to prepare their own plans and specifications, in which case the Reinforced Earth Company will prepare guidelines and assist the agency with these items. A statement covering the proprietary information of the structure is included on each plan sheet.

The role of the supplier is generally handled by a subcontract with the principal contractor on the project or in the same manner as any materials supplier is handled in a contract by the contracting agency. The materials supplied include facing elements, steel or precast concrete, reinforcing strips, fastners and joint materials, but do not include the backfill materials.

The technical assistance supplied prior to and during construction, includes participation in prebid and preconstruction conferences, and during the early stages of construction, on-site assistance to construction personnel.

## Sheet Pile Walls

Design Lengths - In lieu of using soil fill to increase levee heights, sheet pile walls are being considered as floodwalls. Sheet pile walls could be either steel or concrete panels designed to resist the hydrostatic forces of high river flows. Assuming that water could rise to Elevation 14 on the river side of the wall, the following preliminary estimates of sheet pile embedment are provided.

SHEET PILE EMBEDMENT

| Station | Embedment Length | Estimated Embedment |
|---|---|---|
| 10+00 to 20+00 | 14 | Elevation -4 |
| 20+00 to 32+00 | 16 | Elevation -6 |
| 32+00 to 42+00 | 10 | Elevation 0 |

Assumes top of levee at Elevation +10

The sheet pile embedments listed above are, in part, predicated on the assumption that the levee crest at the location of the sheet piles would be positioned at Elevation 10 or higher on both sides of the sheet piling. If dredging in front of the wall is anticipated, the Geotechnical Engineer should be contacted for further studies.

Construction Considerations - Along most of the levee alignment, the sheet piles will be suspended within the Bay Mud deposits. During driving, it may be difficult to stop the sheet pile penetration. Once installed, it is expected that friction will develop between the mud and the sheet pile to support it within an 8-hour period. The Contractor should be advised of the problem and provide a means to hold the sheet pile at its design depth until a sufficient bond (or freeze) develops between the mud and the sheet pile to hold it in place.

Within the area of Station 28+00 to Station 32+00, it is suspected that old concrete floor slabs or concrete rubble are present within the existing levees. Existing levees in other areas between Station 20+00 and 42+00 also are suspected to contain rubble concrete or other debris. In order for the sheet piling to penetrate to design depths, it may be necessary to pre-excavate a trench through the rubble materials.

## Seepage

Seepage is known to occur through the existing levee system at various locations. At the South Bay Yacht Club, seepage was noted at the base of the existing gabion wall. Seepage also occurs in the area of the old cannery building during high tide levels and evidence of seepage also is present near the base of the levee in the vicinity of Catherine Street. These seepage areas are noted on Figure 1.

Most likely, seepage is occurring through pervious zones of the levee which probably contain pockets of rubble fill. Seepage also could be occurring at the interface between buried building foundations or floor slabs and the native soils as well as along the abandoned oil pipelines extending from the Guadalupe River to Hope Street.

It would be extremely difficult to locate the precise areas of seepage in order to intercept them or grout them closed.

Interlocking sheet pile walls could be installed along selected reaches in an attempt to stop leakage. The sheet piles should extend through the existing fills and 3 feet into the underlying native soils. Estimates of depth to native soil are shown on the Idealized Soil profile presented as Figure 2. In areas containing subsurface obstructions, such as buried concrete slabs or foundations, it may be necessary to pre-excavate the soil and rubble fill prior to sheet pile driving. The excavation would require backfilling with sand or lean grout prior to or after the sheet pile is installed.

Alternatively, a slurry trench cut-off could be constructed along the areas of suspected seepage. The slurry trench should be at least 18 inches wide and extend 3 feet into the native soils below existing fills. A slurry of cement and bentonite should be placed in the trench to minimize the potential for seepage.

Where buried piping is encountered, the lines should be excavated and replaced with a relatively impervious soil fill compacted to a minimum degree of compaction of 90 percent. The fill should meet the general requirements described previously in this report for levee fill materials.

### Groundwater Control

As discussed previously, free groundwater was encountered as high as Elevation 5-1/2± within the existing levee section and just below the existing grade in borings drilled near the inboard levee toe. Because of the presence of shallow groundwater and seepage, it should be expected that some means of groundwater control will be needed during the excavation of weak fills and soft Bay Mud from beneath the proposed retaining wall footing locations and during the placement of engineered fill to replace those materials.

High groundwater and the presence of pervious deposits below the water level also pose potential difficulties for the installation of drilled piers. Piers extending through existing levee fills or native sandy soils will most likely require temporary casing to maintain the sides of the excavations and dewatering could be difficult. In the event that dewatering might prove to be ineffective, concrete could be placed using tremmie techniques.

The intent of this discussion is to point out some of the difficulties which should be anticipated during excavation and foundation construction for this project. The Contractor should anticipate the need for dewatering excavations for this project. The proposed dewatering scheme should be reviewed by the Geotechnical Engineer prior to installation in an attempt to identify potential associated problems such as piping by subsurface erosion or settlements.

## LIMITATIONS

The opinions, conclusions and recommendations contained in this report have been based on information obtained from exploratory borings made at widely separated locations, visual reconnaissance of the project alignment, review of available topographic and historic information, and upon local experience and judgment. The locations of the borings were approximately determined by tape measurement from existing site features. Elevations noted on the borings were approximately determined by interpolation between spot elevations shown on available topographic maps. The limits of previous buildings shown on the Site and Exploration Plan were based upon approximate correlations between the conditions shown on a 1957 aerial photograph of the area and those which were present at the time of this study. The locations of the borings, the previous building limits and the elevation data referenced in this report should be considered accurate only to the degree implied by the method used.

The report also has been prepared based upon the alternative floodwall and levee systems submitted to us by Anderson-Nichols & Co., Inc. during the course of this study. It is understood that Woodward-Clyde Consultants will be provided the opportunity to review the design of the selected systems in order to determine whether any change in concept may have an effect on the validity of the recommended geotechnical design parameters.

It should be expected that variations in the subsurface conditions will be encountered during construction. In order to permit correlation between the soil data obtained during this investigation and the actual conditions encountered during construction, it is recommended that the Geotechnical Engineer be retained to provide construction review during the earthwork and foundation construction phases of the work.

# APPENDIX A

# FIELD INVESTIGATION AND LABORATORY TESTS

# GUADALUPE RIVER FLOOD CONTROL IMPROVEMENTS

San Jose, California

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## FIELD INVESTIGATION

Fourteen exploratory borings which ranged in depth from 11-1/2 feet to 45 feet were made for this investigation between the dates of October 20, 1982 and November 3, 1982 under the supervision of a representative of Woodward-Clyde Consultants. Their locations are shown on the Site and Exploration Plan, Figure 1. Borings 1 through 7 were advanced with a 4-inch diameter rotary drill. Drilling "mud" was used to prevent caving and water loss. Borings 8 through 14 were advanced with a 6-inch diameter continuous flight power auger. Soils were visually classified in the field. In-place samples were obtained using two types of drive samplers: the modified California drive sampler and the standard split-barrel sampler. The modified California drive sampler used had 2-inch inside diameter and 2-1/2-inch outside diameter while the standard split-barrel sampler had 1-3/8 inch diameter and 2-inch outside diameter. Each sampler was driven 18 inches into the soil by a 140-lb. hammer free falling 30 inches. The number of blows to penetrate the last 12 inches is shown on the Logs of Borings, Figures 4 through 17.

In addition, samples of the Bay Mud soils were obtained using thin-walled 3 inch diameter Shelby tubes. Shelby tube sampling was accomplished by pushing the tube into the soil with a continuous and rapid motion under hydraulic pressure.

When the samplers were withdrawn from the hole, the samples were carefully removed, sealed to prevent moisture loss, and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing.

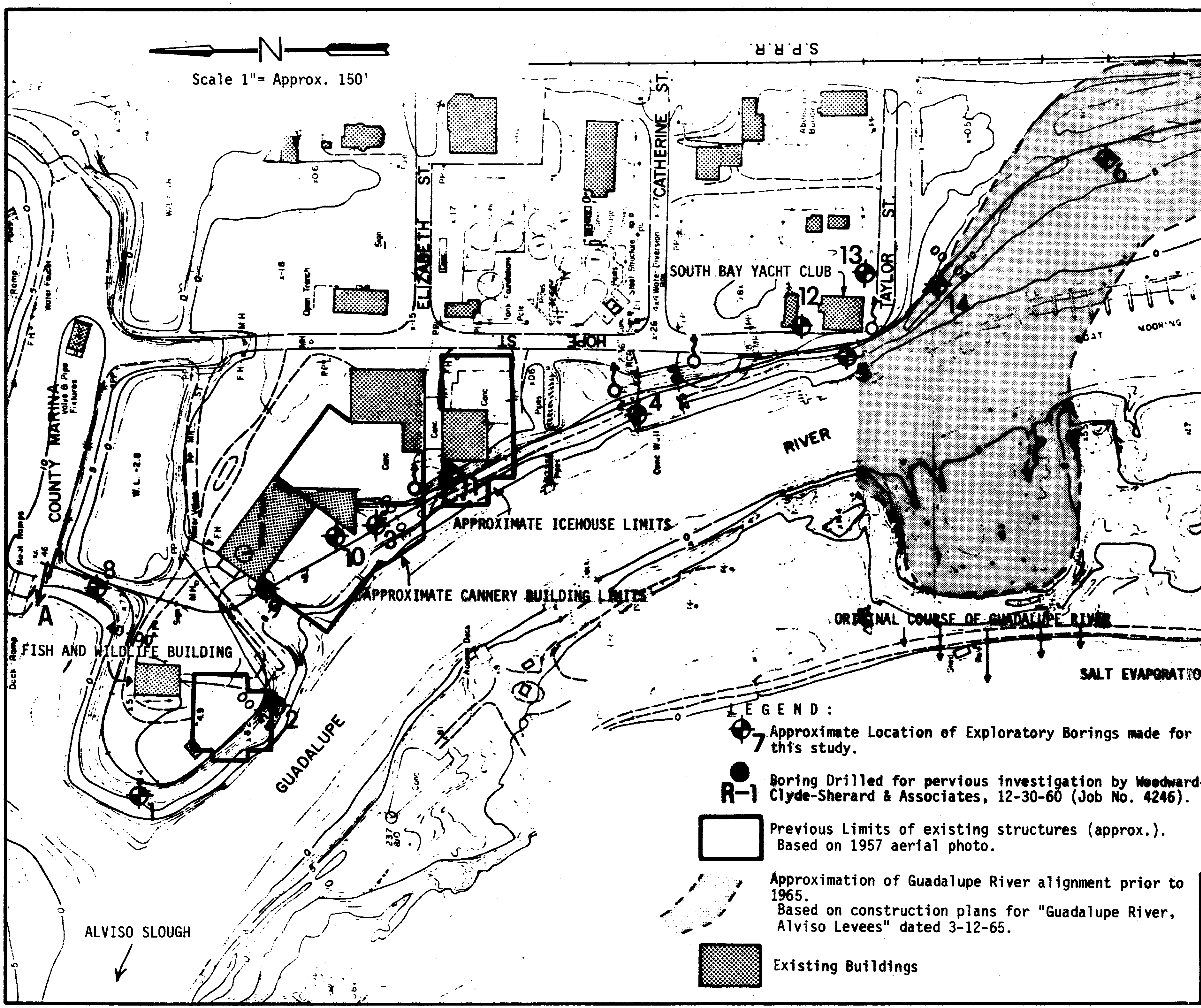
The attached legend, Figure 3, illustrates the notation used on the Logs of Borings for the types of samplers and methods of advancing them.

## LABORATORY TESTS

The water content, dry density and unconfined compressive strength were determined for selected samples in order to estimate the strength and compressibility of the soils. The results of these tests together with the resistance to penetration of the sampler, are shown on the corresponding sample locations on the Logs of Borings.

Grain size distributions were determined for selected samples of the sand and silty sand deposits. The results of these tests are represented graphically on Figures 18 and 19.

Atterberg Limits (Liquid and Plastic Limits) were determined for a sample of clayey sand to aid in the material classification. The results of this test are shown graphically on Figure 20.


N
Scale 1"= Approx. 150'
S.P.R.R.
R-1
A
A
ELIZABETH ST.
CATHERINE ST.
TAYLOR ST.
HOPE ST
SOUTH BAY YACHT CLUB
BOAT MOORING
COUNTY MARINA
RIVER
GUADALUPE
APPROXIMATE ICEHOUSE LIMITS
APPROXIMATE CANNERY BUILDING LIMITS
ORIGINAL COURSE OF GUADALUPE RIVER
SALT EVAPORATION PONDS
FISH AND WILDLIFE BUILDING
ALVISO SLOUGH
1
2
3
4
5
6
7
8
9
10
11
12
13
14
LEGEND:
7 Approximate Location of Exploratory Borings made for this study.
R-1 Boring Drilled for pervious investigation by Woodward-Clyde-Sherard & Associates, 12-30-60 (Job No. 4246).
Previous Limits of existing structures (approx.). Based on 1957 aerial photo.
Approximation of Guadalupe River alignment prior to 1965. Based on construction plans for "Guadalupe River, Alviso Levees" dated 3-12-65.
Existing Buildings
Surface Seepage Observed on 12-2-82

SITE AND EXPLORATION PLAN
Guadalupe River Improvements
San Jose, California

Project No. 15482-V
**Woodward-Clyde Consultants**

Figure 1

N O T E S:

1. SEE FIGURE 1 FOR SECTION AND BORING LOCATIONS
2. THIS IDEALIZED SOIL PROFILE HAS BEEN CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN BORINGS DRILLED AT VARYING SPACINGS AND PROJECTED TO THE SECTION LINE. THE DASHED LINES CONNECTING THE VARIOUS LAYERS AT EACH BORING LOCATION ARE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY AND SHOULD NOT BE CONSTRUED TO REPRESENT THE ACTUAL CONDITIONS IN THE FIELD.
3. FOR DETAILED DESCRIPTIONS OF THE MATERIALS ENCOUNTERED IN EACH OF THE BORINGS DRILLED FOR THIS INVESTIGATION, SEE FIGURES 4 THROUGH 17 .
4. THE EXISTING GRADE INDICATED IS APPROXIMATE AND IS BASED UPON INTERPOLATIONS BETWEEN SPOT ELEVATION DATA SHOWN ON AN AERIAL PHOTO BASE MAP OF THE PROJECT AREA (1982) FURNISHED BY THE SANTA CLARA VALLEY WATER DISTRICT.

L E G E N D:

▨ FILL -- VARIOUS MIXTURES OF CLAYS, SANDS AND ROCK FRAGMENTS WITH INCLUSIONS OF BROKEN ASPHALT, CONCRETE, WOOD AND OTHER DEBRIS.

▩ BAY MUD -- SOFT, COMPRESSIBLE SILTY CLAY WITH TRACES OF ORGANIC MATERIAL.

▥ CLAYS -- GENERALLY STIFF, MEDIUM TO HIGH PLASTICITY SILTY AND SANDY CLAYS.

⛆ SANDS -- GENERALLY MEDIUM DENSE, FINE TO MEDIUM GRAINED SANDS AND SILTY TO CLAYEY SANDS WITH SOME SCATTERED FINE GRAVEL.

∇ HIGHEST WATER LEVEL RECORDED

0 50 100 200 300
HORIZONTAL SCALE IN FEET

VERTICAL EXAGGERATION: 10X

SECTION A - A: IDEALIZED SOIL PROFILE
GUADALUPE RIVER IMPROVEMENTS
San Jose, California

Project No. 15482V
**WOODWARD-CLYDE CONSULTANTS**

Figure 2

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# BORING LOG LEGEND SHEET

Date Drilled:

Type of Boring:

Hammer Weight:

Remarks:

|  |  |  |  | LABORATORY TESTS |  |  |
|---|---|---|---|---|---|---|
| Depth, Ft. | Samples | Blows/Ft. | DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|  | ▨ |  | 2-INCH I.D. MODIFIED CALIFORNIA SAMPLER |  |  |  |
| 5 |  |  |  |  |  |  |
|  | ⊠ |  | 3-INCH O.D. SHELBY TUBE SAMPLER |  |  |  |
| 10 | ▥ |  | 2-INCH O.D. STANDARD SPLIT-SPOON SAMPLER |  |  |  |
| 15 | ☐ | 29 | BLOW COUNT WITH A 140-LB. HAMMER FALLING 30 INCHES (CATHEAD) |  |  |  |
|  | ☐ | 29* | BLOW COUNT WITH A 140-LB. HAMMER FALLING 30 INCHES (WINCH) |  |  |  |
| 20 |  |  |  |  |  |  |
|  | ☐ | P | SAMPLER ADVANCED BY HYDRAULIC PUSHING |  |  |  |
| 25 |  | ▽ | WATER LEVEL MEASURED: |  |  |  |
|  |  | ATD | At Time of Drilling |  |  |  |
|  |  | 3 Hrs. | In Hours or Days After Drilling |  |  |  |
|  |  | 11-2-82 | On Date Indicated |  |  |  |
| 30 |  |  |  |  |  |  |

Proj. No. 15482V

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Figure 3

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 1

Date Drilled: October 20, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.

Remarks: Auger to 5 ft., then rotary with bentonite mud

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +8½±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | **SILTY CLAY FILL (CH)** Poorly compacted, damp, brown and gray, with traces of coarse sand, roots, and zones of fine sand | | | |
| | 1 | 5 | Brown-gray, moist | 32 | 82 | 3215 |
| 5 | | | **SANDY CLAY (CL-SC)** Poorly compacted, moist to wet, gray, with gravel<br>▽ 24 Hrs.<br>▽ 10-26-82 | | | |
| | | | **SAND (SW):** Loose, brown and gray, with fine gravel (FILL) | | | |
| | 2 | 4 | | | | |
| | 3 | 2 | **SILTY CLAY (CH)** Very soft, dark gray to black | 56 | 66 | - |
| 10 | | | (BAY MUD) | | | |
| | | | Black | | | |
| 15 | 4 | 4 | | | | |
| | | | With coarse sand, fine gravel, wood splinters | | | |
| | | | Soft, gray | | | |
| | 5 | 19 | | 33 | 86 | 1364 |
| 20 | | | **SILTY CLAY (CH)** Medium, brown-gray and gray mottled, with some cemented nodules | 22 | 103 | 3941 |
| | | | Stiff | | | |

Proj. No. 15482V

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Figure 4a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 1

(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | 6 | 21 | SILTY CLAY .......continued | | | |
| 25 | | | | | | |
| | | | GRAVELLY SAND (SP-GP)<br>Medium dense, gray | | | |
| | 7 | 21 | SILTY CLAY (CL)<br>Stiff, orange-brown and gray mottled with brown, with fine sand and traces of gravel | | | |
| 30 | | | | | | |
| | | | More sandy | | | |
| | 8 | 20 | SILTY SAND (SW-SM)<br>Medium dense, gray and brown, with fine gravel<br>+#4=7%<br>-#200=12% | | | |
| 35 | 9 | 13 | Silty clay (CL-ML)<br>Sandy silt (ML-SM) | | | |
| | | | SILTY CLAY (CL)<br>Stiff, orange-brown and gray mottled | | | |
| 40 | 10 | 15 | SILTY SAND (SM-SC)<br>Medium dense, brown and gray, with some clay | 22 | 109 | - |
| | 11 | 25 | SILTY CLAY (CL-CH)<br>Very stiff, brown mottled | | | |
| | | | BOTTOM OF BORING @ 43 ft. | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Figure 4b

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 2

Date Drilled: October 21, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.
Remarks: Auger to 15 ft., then rotary with bentonite mud
(See Legend Sheet for sampler types and hammer weights)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS: Moisture Content, % | LABORATORY TESTS: Dry Density, pcf | LABORATORY TESTS: Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | Surface Elevation: +8½± | | | |
| | | | **SILTY GRAVEL FILL (GM)**<br>Reddish-brown, dry to damp, with fragments of broken concrete, slightly clayey | | | |
| | | | Brown to dark brown, with increasing clay, moist<br>(FILL) | | | |
| 5 | | | ▽ 10-26-82 | | | |
| | | | **SANDY CLAY (CH-SC)**<br>Very soft to soft, gray<br>(DREDGED BAY MUD FILL) | | | |
| | | | ▽ ATD | | | |
| | | | **SILTY CLAY (CH)** | | | |
| | 1 ☒ | P | Very soft, black, with wood fragments and lenses of very loose sand (SP-SM) | 44 | 76 | - |
| 10 | | | (BAY MUD) | | | |
| | | | Sand (SP-SM) | | | |
| | | | Massive wood (plywood in sampler tip) | | | |
| 15 | | | | | | |
| | 2 ☒ | P | Stiff | 33 | 89 | 2280 |
| | | | **SILTY CLAY (CH)** | | | |
| | 3 ▨ | 17 | Stiff, dark gray and brown mottled | 28 | 96 | 2630 |
| 20 | | | | | | |
| | | | **SANDY CLAY (CL-SC)**<br>Stiff, orange-brown and olive gray mottled | | | |

Proj. No. 15482V

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Figure 5a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 2

(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| 25 | 4 | 24 | SANDY CLAY .......continued<br>-#200=54% | 28 | 106 | 3120 |
| | 5 | 15 | SAND (SP-SM)<br>Medium dense, brown and gray, uniform and fine<br>+#4=0%<br>-#200=12% | | | |
| 30 | 6 | 15 | | | | |
| | | | Sand (SP), clean, gray | | | |
| 35 | 7 | 15 | With some fine gravel to 3/8-inch<br>+#4=3%<br>-#200=6% | | | |
| 40 | 8 | 23 | | | | |
| | | | SILTY CLAY (CL-CH)<br>Hard, olive-gray and brown mottled, with calcareous nodules | | | |
| 45 | 9 | 53 | | 16 | 117 | 16,370 |
| | | | BOTTOM OF BORING @ 45 ft. | | | |

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 3

Date Drilled: October 21-22, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.
Remarks: Auger to 7 ft., then rotary with bentonite mud
(See Legend Sheet for sampler types and hammer weights)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS | | |
|---|---|---|---|---|---|---|
| | | | | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
| | | | Surface Elevation: +9± | | | |
| | | | **ASPHALT over Base Rock** | | | |
| | | | **SILTY GRAVEL FILL (GM-GC)**<br>Poorly compacted, damp, brown and gray | | | |
| | | | **SILTY CLAY FILL (CH)**<br>Poorly to moderately compacted, damp, brown, with sand, fine gravel, and broken concrete | | | |
| 5 | | | ↓ Sandy, (CL), with rock fragments | | | |
| | | | ▽ 10-26-82 | | | |
| | | | **CLAYEY GRAVEL FILL (GC)**<br>Poorly compacted, brown, with gravel to 3/4-inch | | | |
| 10 | | | ↓ Intermixed rock fragments, sandy zones and silty clay, gray-brown | | | |
| | | | (FILL) ↑ | | | |
| | | | **SILTY CLAY (CH)**<br>Very soft to soft, black | | | |
| 15 | 1 | P | (BAY MUD) | | | |
| | | | ↓ Stiff | 33 | 90 | 2240 |
| | 2 | 19 | | 27 | 97 | 2290 |
| | | | **SILTY CLAY (CH)**<br>Stiff, dark gray and gray-brown mottled | | | |
| 20 | | | **SILTY CLAY (CL-CH)**<br>Stiff, light brown and gray-brown, with fine sand | | | |
| | | | ↓ Grading to Clayey Sand (SC) | | | |
| | 3 | 19 | | | | |

Proj. No. 15482V

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Figure 6a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 3
(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | S A N D (SP) ......continued<br>Medium dense, brown and gray | | | |
| 25 | | | Gray, coarser | | | |
| | | | +#4=16%<br>-#200=6% | | | |
| | 4 | 14 | | | | |
| | | | SILTY CLAY (CL-CH)<br>Stiff, light brown and gray mottled, with fine sand | | | |
| 30 | | | Silty sand lens, (SM) | | | |
| | 5 | 12 | -#200=69% | 20 | 109 | 1364 |
| | | | Less plastic (CL), medium, with interbedded layers of clayey sand | | | |
| 35 | | | SILTY SAND (SM): Medium dense, gray & brown | | | |
| | 6 | 14 | Sandy silt (ML-SM), orange-brown with gray | | | |
| | | | SILTY CLAY (CL)<br>Stiff, orange-brown and gray mottled | | | |
| | | | BOTTOM OF BORING @ 36-3/4 ft. | | | |
| 40 | | | | | | |
| 45 | | | | | | |

| Proj. No. 15482V | **Woodward-Clyde Consultants** | Figure 6b |
|---|---|---|

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 4

Date Drilled: October 22, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.

Remarks: Auger to 7½ ft., then rotary with bentonite mud
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +9±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS<br>Moisture Content, % | <br>Dry Density, pcf | <br>Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY FILL (CH): Poorly compacted, damp to moist, brown, with some sand | | | |
| | | | With rock fragments, sandy zones, and shells | | | |
| | 1 | 8 | | 27 | 86 | - |
| | | | SILTY SAND FILL (SP-SM): Poorly compacted, moist, brown | | | |
| | | | CLAYEY GRAVEL FILL (GC-CL)<br>Poorly compacted, moist to wet, brown | | | |
| 5 | 2 | 6 | SILTY CLAY FILL (CH): Poorly compacted, moist, gray and brown, with gravel | 27 | 95 | 1950 |
| | | | ▽ 10-26-82<br>SANDY CLAY (CH): Medium, moist, dark gray to black, with shell fragments (DREDGED BAY MUD FILL)<br>Soft<br>(FILL) | | | |
| 10 | | | SILTY CLAY (CH)<br>Very soft to soft, dark gray to black, with organic material | | | |
| | 3 | P | (BAY MUD) | 53 | 70 | 470 |
| | 4 | 4 | Sand lens | 48 | 71 | 460 |
| 15 | | | Very silty, (MH) | | | |
| | 5 | 7 | Soft, black | 44 | 75 | 540 |
| | | | SILTY CLAY (CL-CH)<br>Stiff, gray, light gray and brown mottled, with calcareous nodules | | | |
| 20 | 6 | 20 | | 22 | 106 | 3950 |
| | | | SILTY CLAY (CL-CH): Very stiff, orange-brown and gray mottled with brown<br>Sand lens | | | |

Proj. No. 15482V

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Figure 7a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 4

(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY .......continued | | | |
| 25 | 7 | 38 | Hard, with cemented nodules, (CL) | 19 | 110 | 9600 |
| | | | Less plastic (CL), stiff | | | |
| 30 | 8 | 15 | | 21 | 107 | 3120 |
| | | | With fine sand | | | |
| 35 | 9 | 21 | | | | |
| | | | BOTTOM OF BORING @ 37 ft. | | | |
| 40 | | | | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Figure 7b

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 5

Date Drilled: October 22, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.
Remarks: Auger to 7½ ft., then rotary with bentonite mud
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +11±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS: Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | **SANDY GRAVEL FILL (GC-SC)**<br>Moderately compacted, damp, gray, with gray-brown clay, with some cobbles | | | |
| | 1 | 21 | | 5 | 118 | - |
| 5 | 2 | 19 | Moist | 7 | 107 | - |
| | | | **SILTY CLAY FILL (CL-CH)**<br>Poorly to moderately compacted, moist, gray, with gravel, sandstone pieces, and shells<br>(CH) | | | |
| | | | 10-26-82 | | | |
| 10 | 3 | 14 | **CLAYEY GRAVEL FILL (GC)**<br>Poorly compacted, brown, wet, with zones of gray silty sand (SP-SM) | 18 | 107 | - |
| | | | **SILTY CLAY FILL (CH)**<br>Poorly compacted, dark gray, with sand, gravel, and shells<br>(FILL) | | | |
| 15 | 4 | 12 | **SILTY CLAY (CL-CH)**<br>Medium, dark gray to black, very silty | | | |
| | 5 | 11 | (BAY MUD) | | | |
| 20 | 6 | 25* | **SILTY CLAY (CH)**<br>Stiff, dark gray and brown mottled | 27 | 96 | 3700 |
| | | | **SILTY CLAY (CL-CH)**<br>Very stiff, orange-brown and brown mottled with gray | | | |

Proj. No. 15482V

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Figure 8a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 5
(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY .......continued | | | |
| 25 | 7 | 17 | Less plastic (CL), gray-brown and light brown, with fine sand | 19 | 108 | 4140 |
| 30 | 8 | 14 | Orange-brown and gray | | | |
| | | | With coarse sand and fine gravel | | | |
| 35 | 9 | 23 | Stiff, orange-brown and gray-brown mottled with red | 20 | 108 | 3702 |
| | | | BOTTOM OF BORING @ 37 ft. | | | |
| 40 | | | | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Figure 8b

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 6

Date Drilled: November 3, 1982
Type of Boring: 6" Auger/Rotary with Mud
Hammer Weight: 140 lbs.

Remarks: Auger to 6½ ft., then rotary with bentonite mud

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +8±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | **SILTY CLAY FILL (CH)**<br>Poorly compacted, damp, brown-gray, with zones of silty sand (SM), roots, and wood | | | |
| | 1 | 4 | Wet<br>11-8-82 | 44 | 75 | 930 |
| | | | ▽<br>**SILTY CLAY FILL (CH)**<br>Poorly compacted, dark gray to black, with brown sand intermixed | | | |
| 5 | 2 | 3 | | 62 | 62 | - |
| | | | **SILTY CLAY (CH)**<br>Soft, dark gray and black, with seams and lenses of fine sand throughout layer, with some organic material<br>(DREDGED BAY MUD FILL)<br>Sand (SP), black | | | |
| | | | Glass fragment noted in cuttings | | | |
| 10 | 3 | 5 | | 54 | 68 | 630 |
| 15 | 4 | 6 | With high concentration of organic material<br>Sand (SP) | 31 | 87 | - |
| 20 | 5 | 5 | With wood fragments | 57 | 64 | 880 |

Proj. No. 15482V

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Figure 9a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 6
(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | Black, with gray lenses of silty clay (CL (CL-ML) | | | |
| 25 | | | With gravel and fine sand | | | |
| | 6 | 7 | | 21<br>68 | 106<br>- | -<br>- |
| | | | Very soft, black | | | |
| | | | Soft, with coarse sand and fine gravel | | | |
| 30 | | | Silty sand (SM-SP) | | | |
| | 7 | 8 | Alternating layers of silty clay (CL-ML) to (CH) and silty sand (SM-SP) | 61 | 61 | 780 |
| 35 | | | | | | |
| | 8 | 10 | With some coarse sand and cemented nodules in a clay matrix; fragments of coal and ceramic tile also noted<br>+#4=26%<br>-#200=26% | | | |
| | | | (FILL) | | | |
| | | | Estimated Bottom of Dredged Fill | | | |
| 40 | | | SAND (SP) | | | |
| | 9 | 44 | Dense, gray and brown, with trace gravel | | | |
| | | | BOTTOM OF BORING @ 41½ ft. | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Figure 9b

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 7

Date Drilled: November 3, 1982

Type of Boring: 6" Auger/Rotary with Mud

Hammer Weight: 140 lbs.

Remarks: Auger to 6½ ft., then rotary with bentonite mud

(See Legend Sheet for sampler types and hammer weights)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | Surface Elevation: +10± | | | |
| | | | **SILTY CLAY FILL (CH)**<br>Poorly compacted, moist, gray, with orange and brown staining and wood fragments | | | |
| | 1 ▨ | 6 | --Wet | 58 | 64 | 2000 |
| | | | ▽ 11-8-82 | | | |
| 5 | 2 ▨ | 4 | **SILTY CLAY FILL (CH)**<br>Soft, dark gray, with wood and brown silty sand | 62 | 60 | 930 |
| | | | **SILTY CLAY (CH)**<br>Soft, black to dark gray, with thin sand lenses<br>(DREDGED BAY MUD FILL) | | | |
| 10 | 3 ▨ | 7 | } Sand (SP) | 33 | 86 | - |
| 15 | 4 ▨ | 4 | With traces of shells, wood<br>} Black sand lens | 47 | 72 | 540 |
| 20 | 5 ▨ | 6 | Alternating layers of gray Silty Clay (CL-CH), black Silty Clay (CH), with Sand lenses, (SP), with some wood | 55 | 65 | 780 |

Proj. No. 15482V

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Figure 10a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 7
(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| 25 | | | Sandy Gravel lens (GP), medium dense, black | | | |
| | 6 | 8 | | 52 | 68 | 730 |
| | | | Very soft, with gravel | | | |
| | | | Becoming soft with sand lenses | | | |
| 30 | | | | | | |
| | 7 | 12 | Grading to sandy silt (ML) | | | |
| | | | SAND (SP): Medium dense, gray to black, with wood fragments | | | |
| | | | Becoming gravelly | | | |
| | | | (FILL) | | | |
| | | | Estimated Bottom of Dredged Fill | | | |
| 35 | | | SANDY CLAY (CL)<br>Stiff, orange-brown and gray mottled, with some fine sand, some lime streaks | | | |
| | 8 | 21 | | | | |
| | | | Grading to Sand (SP-SM), medium dense | | | |
| | | | With interbedded layers of sandy clay (CL) | | | |
| 40 | | | SANDY SILT (ML-SM)<br>Medium dense to dense, gray | | | |
| | 9 | 30 | | | | |
| | | | BOTTOM OF BORING @ 41½ ft. | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Figure 10b

**Project:** GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 8

**Date Drilled:** October 25, 1982
**Type of Boring:** 6" Auger
**Hammer Weight:** 140 lbs.
**Remarks:**
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +10½±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY FILL (CL-ML)<br>Moderately compacted, damp, red-brown | 8 | 106 | 1364 |
| | 1 | 16 | SILTY CLAY FILL (CL-CH)<br>Moderately compacted, moist, gray-brown and gray, with brick fragments | 21 | 103 | - |
| 5 | 2 | 23 | SILTY CLAY FILL (CL)<br>Moderately to well compacted, moist, orange-brown, tan, and brown, with rock fragments | 19 | 112 | 7060 |
| 10 | 3 | 17 | SILTY CLAY FILL (CH)<br>Stiff, moist, dark gray and black, with shell fragments and organic material | 33 | 86 | 3020 |
| | | | ↓ Dark gray and gray-brown | | | |
| 15 | 4 | 21 | ▽ 10-26-82 | 32 | 87 | 2490 |
| | | | (FILL) ↑ | | | |
| 20 | 5 | 11 | SILTY CLAY (CH)<br>Medium, black, with organic material<br>(BAY MUD) | 37 | 83 | 1510 |
| | | | SILTY CLAY (CL-CH)<br>Stiff, dark gray with brown | | | |

Proj. No. 15482V

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Figure 11a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 8

(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY ........continued | | | |
| 25 | 6 | 17 | SILTY CLAY (CL)<br>Stiff, gray with orange and brown, with some lime streaks and nodules | 23 | 102 | 3410 |
| | | | --Brown and gray | | | |
| 30 | 7 | 18 | | | | |
| | | | }Silty Sand (SM), brown and gray | | | |
| 35 | 8 | 12 | SANDY SILT (ML)<br>Medium dense, gray, with fine sand lenses | | | |
| | 9 | 8 | SILTY CLAY (CL-ML)<br>Medium, gray | | | |
| 40 | 10 | 21 | Becoming more plastic (CL), stiff, gray with brown | 23 | 102 | 2050 |
| | | | BOTTOM OF BORING @ 42 ft. | | | |
| 45 | | | | | | |

Proj. No. 15482V

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Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 9

Date Drilled: October 25, 1982
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.

Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +7½±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS<br>Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | HETEROGENEOUS FILL<br>Poorly compacted, damp, brown and gray, comprised of a mixture of soils, rock fragments, and broken concrete and asphalt | | | |
| | 1 | 27 | Moist | 5 | 128 | - |
| 5 | 2 | 28 | With high asphalt content | | | |
| | | | ▽ 10-26-82 | | | |
| | | | SANDY CLAY FILL (CH)<br>Very soft to soft, dark gray, with silt | | | |
| 10 | | | }-4" concrete: Old building slab? (FILL) | | | |
| | | | SILTY CLAY (CH)<br>Medium, dark gray and black mottled | | | |
| | 3 | 17 | | 33 | 89 | 1270 |
| 15 | | | BOTTOM OF BORING @ 14 ft. | | | |
| 20 | | | | | | |

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 10

Date Drilled: October 25, 1982
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | Surface Elevation: +8± | | | |
| | | | **HETEROGENEOUS FILL**<br>Poorly compacted, damp to moist, brown containing brick, sand, rock, and concrete | | | |
| | 1 | 15 | | 10 | 98 | - |
| | | | Brown-gray, with broken asphalt | | | |
| 5 | | | | | | |
| | 2 | 19 | | 20 | 103 | - |
| | | | **SILTY CLAY FILL (CL)**<br>Poorly to moderately compacted, moist, brown-gray | | | |
| | | | **SILTY CLAY FILL (CH)**<br>▽ 10-26-82<br>Poorly compacted, wet, dark gray to black, with fine sand | | | |
| 10 | 3 | 25/2" | | 43 | - | - |
| | | | 4" concrete: Old building slab?<br>over brick, shells, rock fragments in clay matrix<br>(FILL) | | | |
| | 4 | 18 | **SILTY CLAY (CL-CH)**<br>Very stiff, dark gray mottled with black, with traces of organic material | 29 | 92 | 4870 |
| 15 | | | | | | |
| | | | **SILTY CLAY (CL-CH)**<br>Very stiff, brown and gray mottled, with cemented nodules | | | |
| | 5 | 30 | | 23 | 105 | 5940 |
| | | | BOTTOM OF BORING @ 18½ ft. | | | |
| 20 | | | | | | |

Proj. No. 15482V

**Woodward-Clyde Consultants**

Figure 13

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 11

Date Drilled: October 25, 1982
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: -1±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS: Moisture Content, % | LABORATORY TESTS: Dry Density, pcf | LABORATORY TESTS: Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | ▽ 10-26-82<br>SILTY CLAY FILL (CH)<br>Poorly compacted, wet, brown-gray, with concrete, brick, rock fragments<br>(FILL) | | | |
| | 1 | 8 | SILTY CLAY (CH)<br>Medium, black to dark gray, with organic material (BAY MUD)<br>--Very silty, soft | 34 | 86 | - |
| 5 | 2 | 15 | SILTY CLAY (CH)<br>Stiff, dark gray-brown and black mottled, with wood fragments | 25 | 100 | 3700 |
| 10 | 3 | 24 | SILTY CLAY (CL-CH)<br>Stiff, orange-brown and gray mottled<br>--Very stiff | 19 | 112 | 5410 |
| | | | BOTTOM OF BORING @ 11½ ft. | | | |
| 15 | | | | | | |
| 20 | | | | | | |

Proj. No. 15482V

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Figure 14

**Project:** GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 12

**Date Drilled:** October 26, 1982
**Type of Boring:** 6" Auger
**Hammer Weight:** 140 lbs.

**Remarks:**

(See Legend Sheet for sampler types and hammer weights)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION<br>Surface Elevation: -1± | LABORATORY TESTS<br>Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY FILL (CH)<br>▽ 6 Hrs.<br>Poorly compacted, moist to wet, brown-gray, with shells, rock fragments and coarse sand | | | |
| | 1 | 6 | SILTY SAND (SP-SM)<br>Loose, gray and brown<br>-#200=12% | | | |
| 5 | 2 | 15 | SILTY CLAY (CH)<br>Medium, gray to dark gray, with fine sand<br>(BAY MUD)<br>Stiff, decreasing sand content | 29 | 95 | 3600 |
| | | | SILTY CLAY (CH)<br>Stiff, gray with brown stains | | | |
| 10 | 3 | 26 | SILTY CLAY (CL-CH)<br>Very stiff, orange-brown and gray mottled with brown, with lime streaks<br>Brown and light brown, stiff | 21 | 108 | 5410 |
| 15 | 4 | 13 | SANDY CLAY (CL-CH)<br>Medium, light brown mottled with orange and black<br>-#200=59%<br>Becoming light brown and brown | 21 | 107 | 1310 |
| 20 | 5 | - | SILTY SAND (SM)<br>Loose, light brown | | | |
| | 6 | 12 | SILTY CLAY (CL-CH)<br>Medium, gray and orange brown mottled with gray | 29 | 94 | 1170 |

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 12
(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY .........continued | | | |
| 25 | | | Stiff | | | |
| | 7 | 32 | Very stiff, with tan | 19 | 110 | 5160 |
| 30 | | | | | | |
| | 8 | 29 | | | | |
| 35 | | | BOTTOM OF BORING @ 35 ft. | | | |
| 40 | | | | | | |
| 45 | | | | | | |

| | | |
|---|---|---|
| Proj. No. 15482V | Woodward-Clyde Consultants | Figure 15b |

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 13

Date Drilled: October 26, 1982
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: -1±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SILTY CLAY FILL (CH): Poorly compacted, moist, brown-gray, with shells & rock fragments (FILL) | | | |
| | | | SILTY CLAY (CH): Soft to medium, moist, dark gray, with organic material (BAY MUD) | | | |
| | 1 | 7 | Sand lens | 33 | 87 | 970 |
| | | | SILTY CLAY (CH)<br>Medium, moist, dark gray with brown, with roots<br>▽ 10-26-82 | | | |
| 5 | 2 | 15 | Stiff | 30 | 92 | 2920 |
| 10 | 3 | 17 | SILTY CLAY (CL-CH)<br>Stiff, moist, light brown, orange-brown and gray, with nodules to 3/8-inch | 19 | 109 | 3605 |
| 15 | 4 | 20 | Medium, (CL-ML) | | | |
| | | | SILTY SAND (SP-SM)<br>Medium dense, gray and brown, fine-grained | | | |
| 20 | | | | | | |
| | 5 | 17 | SILTY CLAY (CL-CH)<br>Stiff, light brown and gray mottled | 18 | 113 | 3310 |
| | 6 | 12 | CLAYEY SAND (SC-SP)<br>Medium dense, light brown and gray mottled<br>PI=10<br>-#200=36%<br>Grading coarser | | | |

Proj. No. 15482V

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Figure 16a

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 13

(Continued)

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|---|
| | | | Grading coarser | -#200=25% | | | |
| 25 | | | | | | | |
| | | | **SILTY CLAY (CL)**<br>Stiff, brown, with fine sand | | | | |
| | 7 | 13 | Sand and fine gravel | | 27 | 98 | 830 |
| | | | Silty sand (SM) | | | | |
| 30 | | | Grading to clayey silt (ML) | | | | |
| | 8 | 22 | **SILTY SAND (SM)**<br>Medium dense, brown and gray | | | | |
| 35 | 9 | 18 | | | | | |
| | | | Grading to sandy silt (ML-SM) | | | | |
| | | | With some clay | | | | |
| 40 | | | | | | | |
| | 10 | 17 | Becoming clayey silt (ML-CL), with fine sand, medium dense | | | | |
| | | | BOTTOM OF BORING @ 42 ft. | | | | |
| 45 | | | | | | | |

Proj. No. 15482V

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Figure 16b

Project: GUADALUPE RIVER IMPROVEMENTS
San Jose, California

# Log of Boring No. 14

Date Drilled: November 3, 1982
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: +11½±

| Depth, Ft. | Samples | Blows/Ft. | MATERIAL DESCRIPTION | LABORATORY TESTS: Moisture Content, % | LABORATORY TESTS: Dry Density, pcf | LABORATORY TESTS: Unconfined Compressive Strength, psf |
|---|---|---|---|---|---|---|
| | | | SANDY SILT FILL (ML-SM)<br>Poorly compacted, damp, brown, with inclusions of Silty Clay (CH) and some shell fragments | | | |
| | 1 | 12 | | 31 | 83 | - |
| | | | SILTY CLAY FILL (CH)<br>Poorly compacted, moist, gray, with brick, organics, and rock fragments | | | |
| 5 | 2 | 12 | ATD<br>With lenses of silty sand (SM) | 35 | 83 | 1220 |
| | | | Sandy zone (SP-SM) | | | |
| 10 | 3 | 10 | Sand lens | | | |
| | | | Sand lens | | | |
| | | | Moderately compacted | | | |
| 15 | | | (FILL) | | | |
| | | | SILTY CLAY (CL-CH)<br>Stiff, gray and dark gray mottled | | | |
| | | | BOTTOM OF BORING @ 18½ ft. | | | |
| 20 | | | | | | |

Proj. No. 15482V

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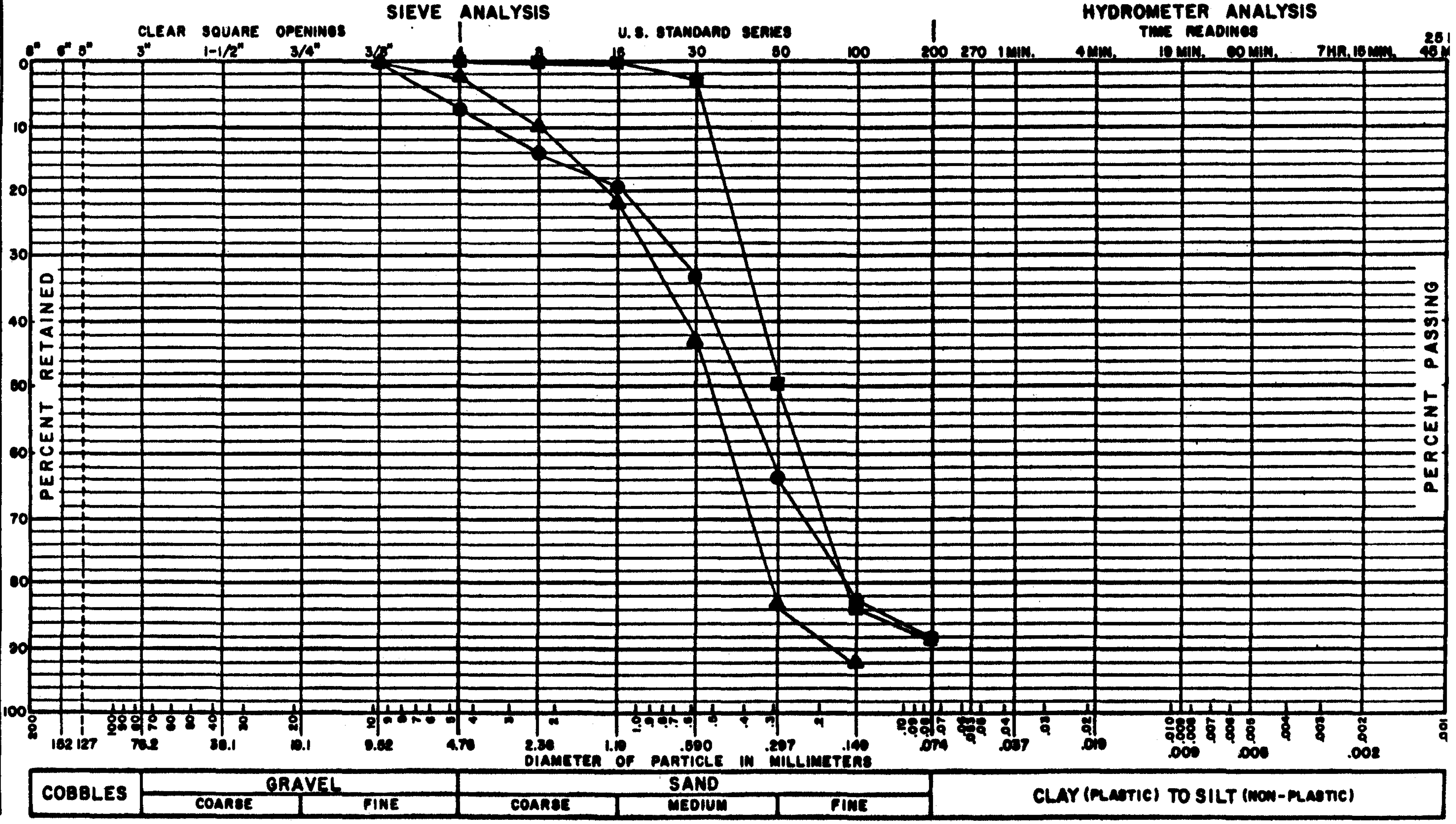
Figure 17

Project No. 15482-V
**Woodward-Clyde Consultants**

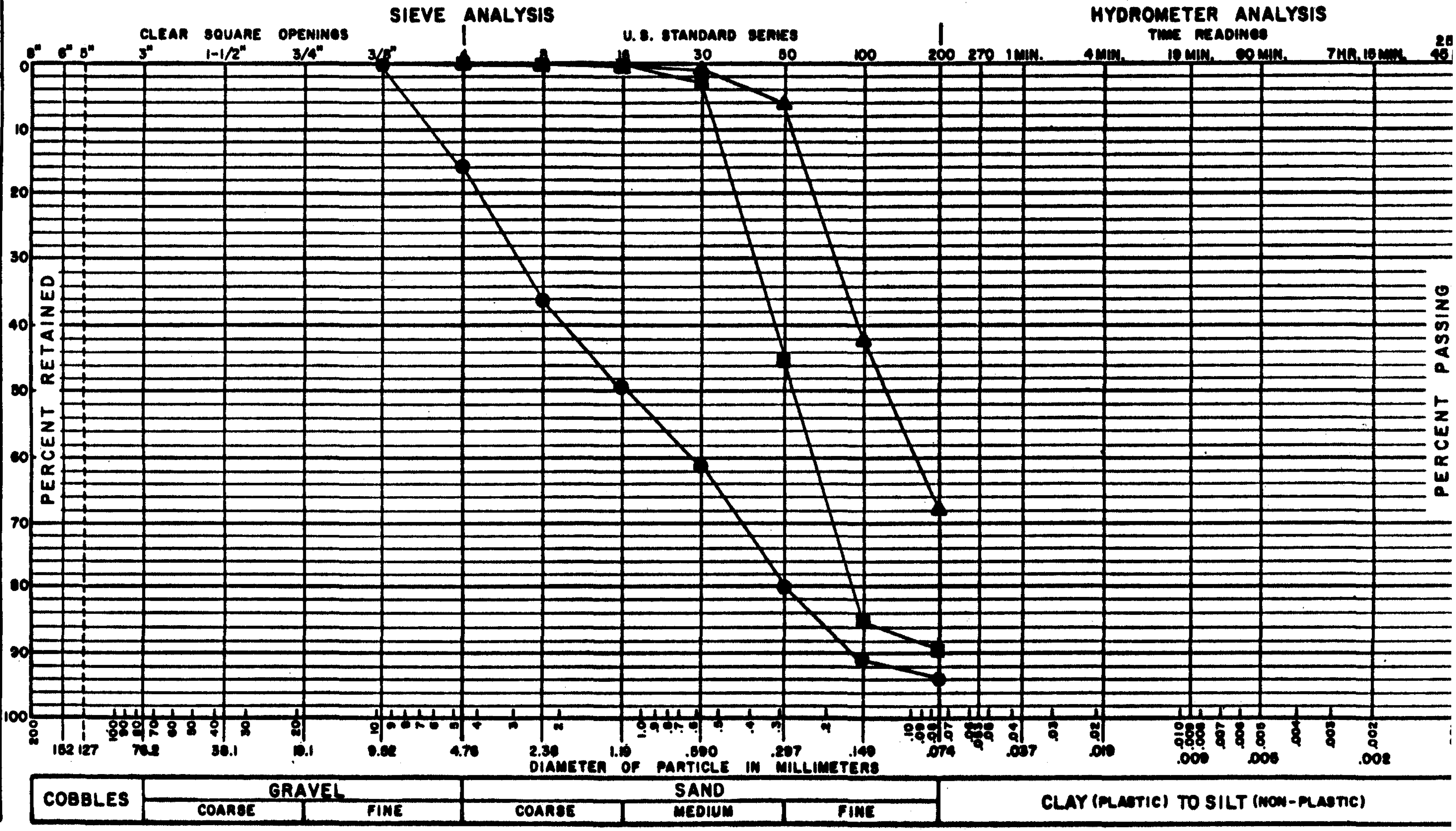
## GRAIN SIZE DISTRIBUTION CURVES
GUADALUPE RIVER IMPROVEMENTS

Figure 18

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| 1-8 | ● | 34.5 ft. | | | SILTY SAND (SW-SM): Gray and brown |
| 2-5 | ■ | 30.0 ft. | | | SAND (SW-SM): Brown and gray |
| 2-7 | ▲ | 35.0 ft. | | | SAND (SP-SM): Brown and gray |



| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| 3-4 | ● | 27.0 ft. | | | SAND (SP-SM): Gray |
| 12-1 | ■ | 3.0 ft. | | | SILTY SAND (SP-SM): Gray and brown |
| 13-6 | ▲ | 24.0 ft. | | | CLAYEY SAND (SP-SC): Brown and gray |



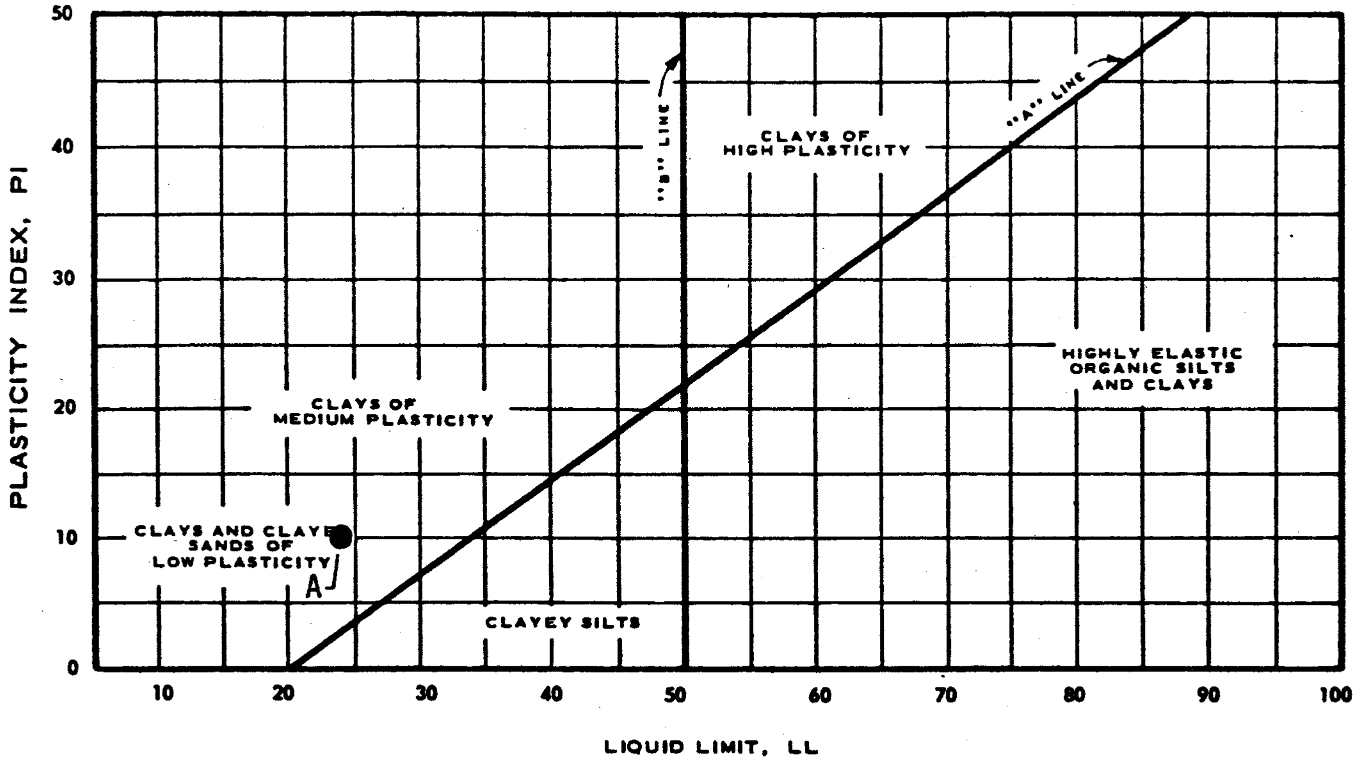
Project No. 15482-V

**Woodward-Clyde Consultants**

## GRAIN SIZE DISTRIBUTION CURVES

GUADALUPE RIVER IMPROVEMENTS

Figure 19



## CLASSIFICATION TEST RESULTS

| SAMPLE IDENTIFICATION | | | ATTERBERG LIMITS | | | GRAIN SIZES – % DRY WEIGHT | | | |
|---|---|---|---|---|---|---|---|---|---|
| LETTER DESIG'N | SAMPLE NO. | DEPTH, FT. | LIQUID LIMIT | PLASTICITY INDEX | PLASTIC LIMIT | SAND | SILT | CLAY | COLLOIDAL |
| A | 13-5 | 22 | 24 | 10 | 14 | 64 | 36 | | |

Project No. 15482-V
WOODWARD-CLYDE CONSULTANTS

PLASTICITY CLASSIFICATION

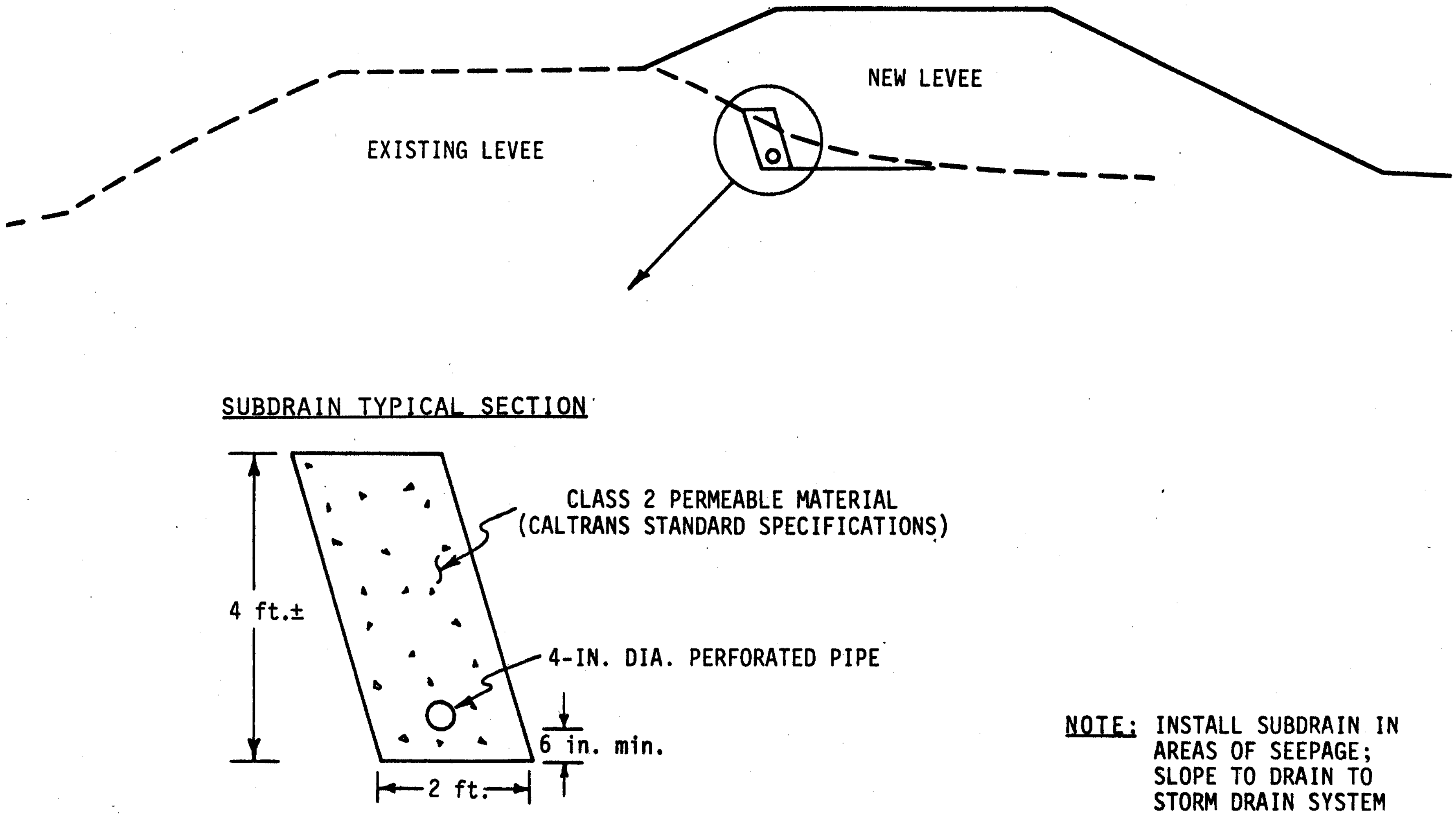
Figure 20

Project No. 15482-V

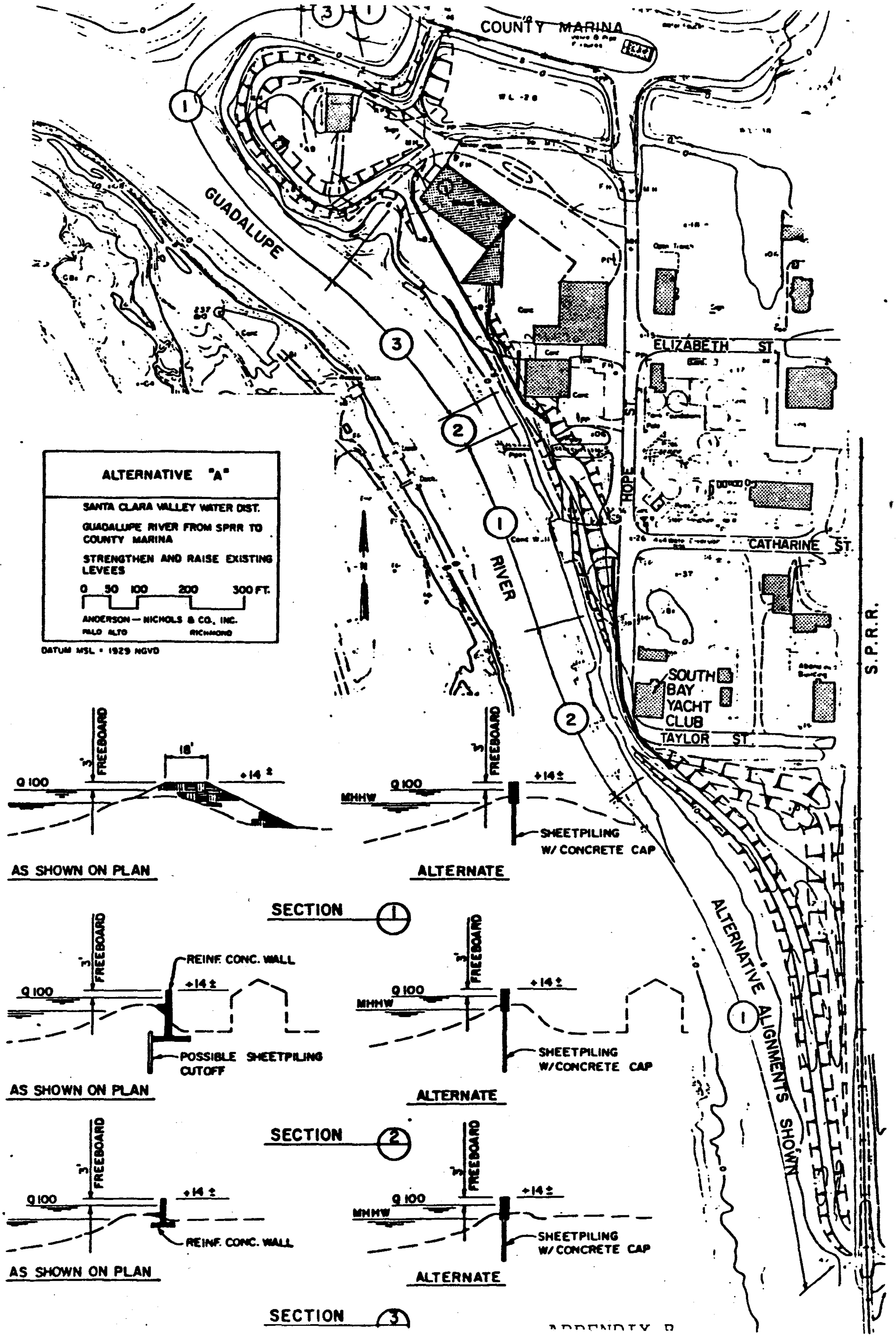
**Woodward-Clyde Consultants**

GUADALUPE RIVER IMPROVEMENT SUBDRAINS
San Jose, California

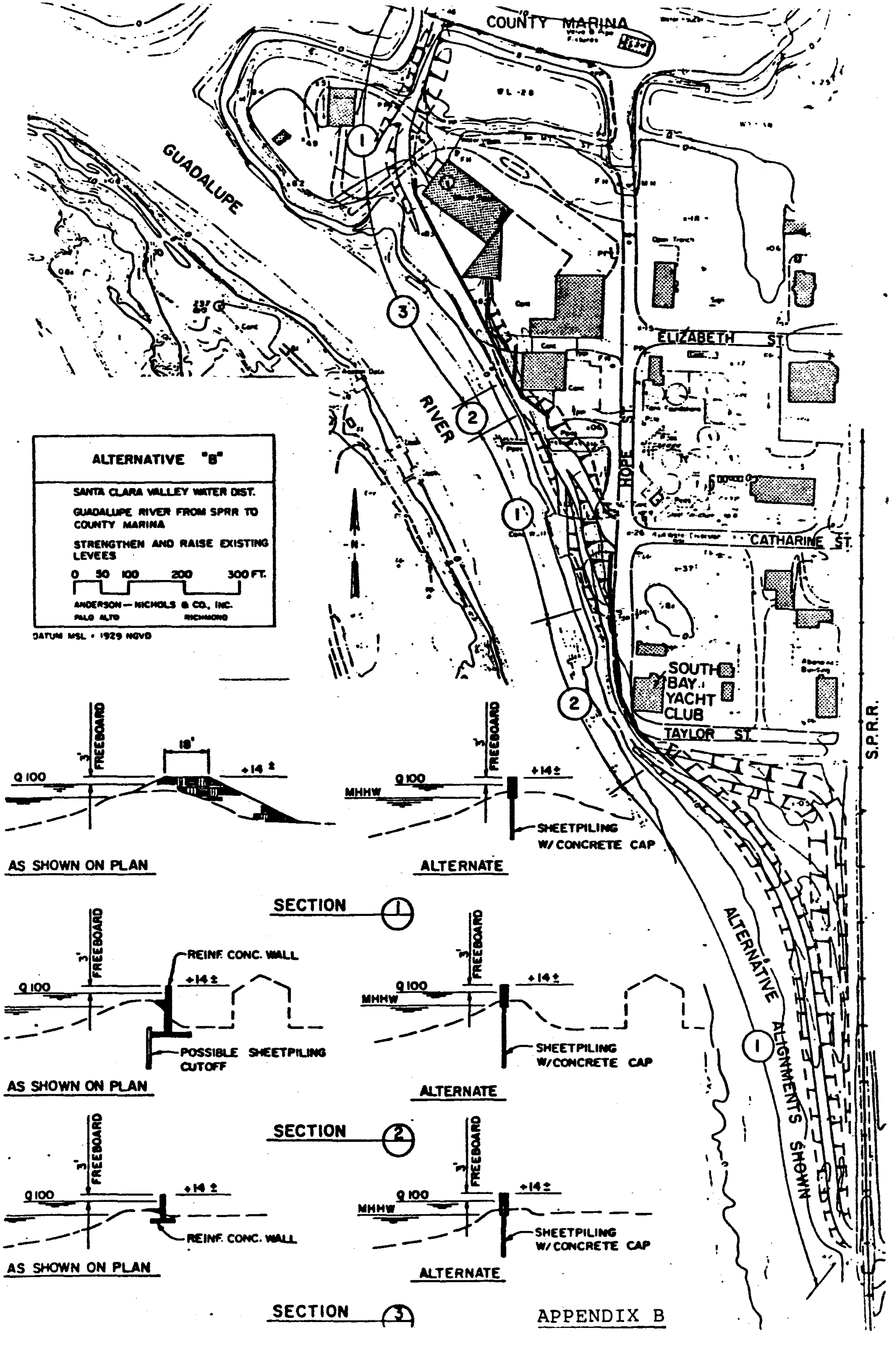
Figure 21


NEW LEVEE
EXISTING LEVEE
SUBDRAIN TYPICAL SECTION
CLASS 2 PERMEABLE MATERIAL
(CALTRANS STANDARD SPECIFICATIONS)
4 ft.±
4-IN. DIA. PERFORATED PIPE
6 in. min.
2 ft.

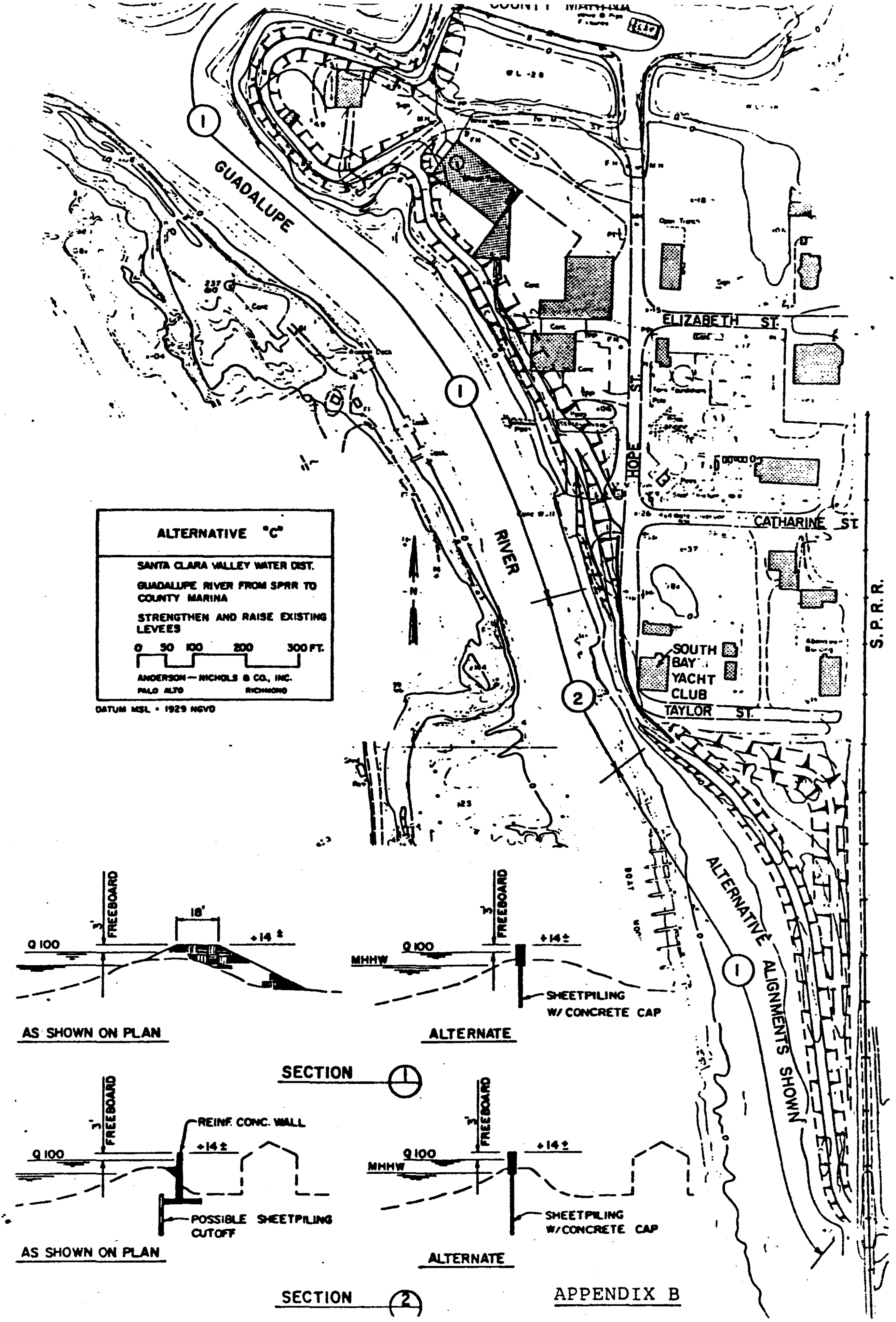
NOTE: INSTALL SUBDRAIN IN AREAS OF SEEPAGE; SLOPE TO DRAIN TO STORM DRAIN SYSTEM


COUNTY MARINA
GUADALUPE RIVER
ELIZABETH ST
HOPE ST
CATHARINE ST
TAYLOR ST
SOUTH BAY YACHT CLUB
S.P.R.R.
ALTERNATIVE ALIGNMENTS SHOWN
ALTERNATIVE "A"
SANTA CLARA VALLEY WATER DIST.
GUADALUPE RIVER FROM SPRR TO COUNTY MARINA
STRENGTHEN AND RAISE EXISTING LEVEES
0 50 100 200 300 FT.
ANDERSON — NICHOLS & CO., INC.
PALO ALTO RICHMOND
DATUM MSL • 1929 NGVD
FREEBOARD
3'
18'
Q 100
MHHW
+14 ±
SHEETPILING W/ CONCRETE CAP
REINF. CONC. WALL
POSSIBLE SHEETPILING CUTOFF
AS SHOWN ON PLAN
ALTERNATE
SECTION ①
SECTION ②
SECTION ③

APPENDIX B


COUNTY MARINA
GUADALUPE
RIVER
ELIZABETH ST.
HOPE
CATHARINE ST.
SOUTH BAY YACHT CLUB
TAYLOR ST.
S.P.R.R.
ALTERNATIVE ALIGNMENTS SHOWN
1
2
3
1'
2
1
N
ALTERNATIVE "B"
SANTA CLARA VALLEY WATER DIST.
GUADALUPE RIVER FROM SPRR TO COUNTY MARINA
STRENGTHEN AND RAISE EXISTING LEVEES
0 50 100 200 300 FT.
ANDERSON — NICHOLS & CO., INC.
PALO ALTO RICHMOND
DATUM MSL • 1929 NGVD
Q 100
3'
FREEBOARD
18'
+14 ±
AS SHOWN ON PLAN
MHHW
Q 100
3'
FREEBOARD
+14 ±
SHEETPILING W/ CONCRETE CAP
ALTERNATE
SECTION 1
REINF. CONC. WALL
Q 100
3'
FREEBOARD
+14 ±
POSSIBLE SHEETPILING CUTOFF
AS SHOWN ON PLAN
Q 100
MHHW
3'
FREEBOARD
+14 ±
SHEETPILING W/CONCRETE CAP
ALTERNATE
SECTION 2
Q 100
3'
FREEBOARD
+14 ±
REINF. CONC. WALL
AS SHOWN ON PLAN
Q 100
MHHW
3'
FREEBOARD
+14 ±
SHEETPILING W/CONCRETE CAP
ALTERNATE
SECTION 3

APPENDIX B


COUNTY MARINA
GUADALUPE
RIVER
①
②
ELIZABETH ST.
CATHARINE ST.
TAYLOR ST.
HOPE ST.
SOUTH BAY YACHT CLUB
S.P.R.R.
ALTERNATIVE ALIGNMENTS SHOWN
N
ALTERNATIVE "C"
SANTA CLARA VALLEY WATER DIST.
GUADALUPE RIVER FROM SPRR TO COUNTY MARINA
STRENGTHEN AND RAISE EXISTING LEVEES
0 50 100 200 300 FT.
ANDERSON — NICHOLS & CO., INC.
PALO ALTO RICHMOND
DATUM MSL • 1929 NGVD
Conc
Q 100
FREEBOARD
3'
18'
+14 ±
AS SHOWN ON PLAN
MHHW
SHEETPILING W/ CONCRETE CAP
ALTERNATE
SECTION ①
REINF. CONC. WALL
POSSIBLE SHEETPILING CUTOFF
SECTION ②

APPENDIX B